JULY 1959
No. 1
V. 31

JOURNAL
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G. E. TROXELL, in "The Resistance to Fire of a Prestressed Concrete Floor Panel," presents a comparison of the general fire resistance features of conventional reinforced and prestressed concrete.

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ROY W. CARLSON and DONALD P. THAYER present a method of placing long monoliths without the occurrence of tensile stress in "Surface Cooling of Mass Concrete to Prevent Cracking."

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A unique foundation is described by IGNACIO MARTIN and SIXTO RUIZ in "Folded Plate Raft Foundation for 24-Story Building."

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"General Elastic Analysis of Flat Slabs and Plates," by JOHN F. BROTCHIE describes a method for the general elastic analysis of moments, shear, and deflection in plates.

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ISRAEL ROSENTHAL reports on "Experimental Investigation of Flat Plate Floors" carried out on circular slabs loaded centrally and eccentrically and tested to destruction.

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A. M. NEVILLE presents "Creep Recovery of Mortars Made with Different Cements" which presents data on the instantaneous recovery and the creep (gradual) recovery of mortars.

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On the Cover—Crossroads Restaurant between Dallas and Fort Worth. A hyperbolic paraboloid shades the entrance and similar concrete shapes form the interior roof area. With their lower surfaces exposed, the paraboloids are supported on slender columns so that no bearing walls are required. Exterior of roof has a built-up cover of tar and felt, without gravel. Plastic covered ports in the paraboloid allow the entry of natural light.

Felix Candela, Mexico City, who has used the hyperbolic paraboloid in Mexico since 1939, was an associate of the architectural firm in charge of construction. Associated Architects and Planners, Dallas, who were responsible for the restaurant and are planners for the surrounding industrial district, includes O'Neil Ford and Richard Colley of Corpus Christi; Sam B. Zisman, San Antonio; and Arch B. Swank, Jr., Dallas. Owner is the Great Southwest Corp.

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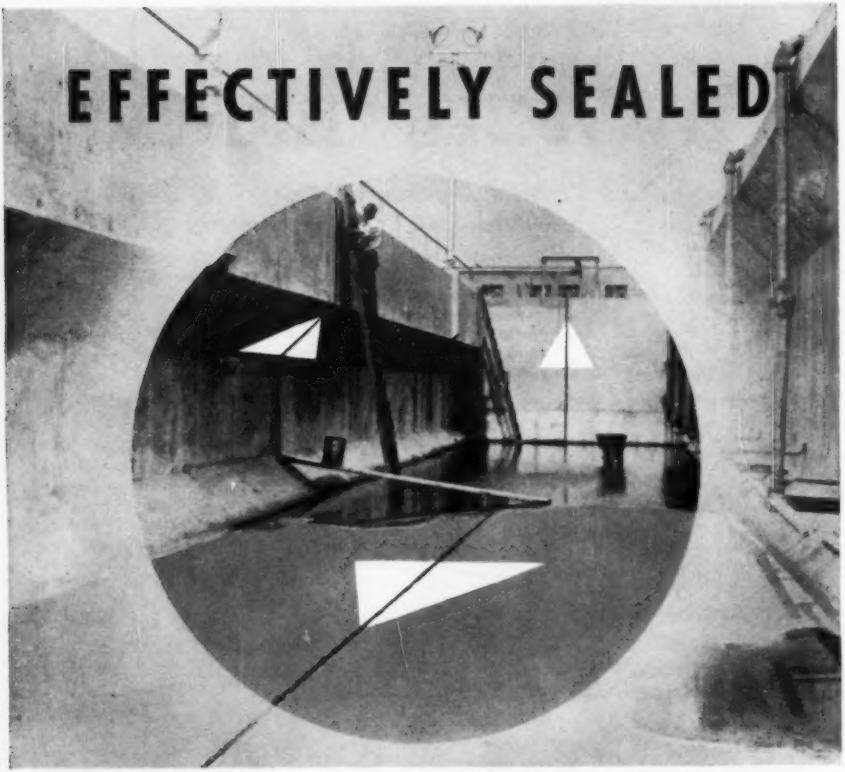

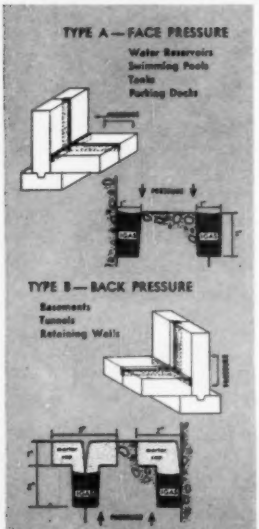

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Title No. 56-4

Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams

PHIL M. FERGUSON and FARID N. MATLOOB

Cutting off reinforcing steel bars in tension zones is shown to lower the shear strength of beams substantially, whereas bending up of bars shows no such ill effect. Exploratory study indicates that both bond stress and diagonal tension are acting to bring about the reduced strengths.

■ THAT THE SIMPLE CHOICE between cutting off reinforcing bars and bending them up can seriously influence the ultimate strength of a beam is not generally appreciated by structural engineers. This gap in understanding probably results from a research emphasis which has centered separately on diagonal tension (shear) failures and on bond strength and has avoided as much as possible the interrelationships between these two. Thus, it came somewhat as a surprise to the authors to find: (1) that where bars were cut off, shear failures were prone to develop prematurely; and (2) that these could scarcely be classified either as bond or as diagonal tension failures but rather appeared to be the result of an interaction between these two factors.

This beam behavior should not have been so unexpected. Research planning quite generally requires the use of a simplified specimen, or even an unusually proportioned specimen, to eliminate undesired complications and to stay within the budgeted time and funds. The information thus obtained may rationally be combined into good design practice. However, one must be ever alert if he is not to overlook fundamental differences between the structure and the test specimen. Hence, as research knowledge accumulates, it becomes increasingly necessary to test specimens which are more complex or more sophisticated.

The practical procedure of stopping or bending up the tension reinforcement introduces new boundaries to the shear problem. With full length steel of uniform area, long span beams fail in moment unless heavily reinforced. Thus, attempts to correlate percent of steel with shear strength are handi-

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●

ACI member FARID N. MATLOOB is chief engineer of the municipality of Mosul, Iraq. He received his BS in civil engineering in Baghdad in 1951, and after graduation worked for the Iraq Ministry of Development. In 1955 he received an Iraq government scholarship to study at the University of Texas, where he was awarded MS degrees in architectural engineering and in civil engineering in 1957. Following employment with the Bureau of Engineering Research at the University of Texas he returned to his present post in Iraq.

capped by two situations: (1) high percentages of steel imply low steel stresses in the intermediate zone between the reaction and load where shear failure is least restrained and most apt to occur; and (2) low percentages of steel tend to be associated only with small shear spans where there is little, if any, intermediate zone far enough removed from both reaction and load to be entirely unrestrained by local vertical compressive stresses. Thus, nearly all reported strengths have tended, from one cause or another, to be optimistically high.

When bars are cut off or bent up, it is possible to have small percentages of steel far removed from the local effects of the load and, in the case of continuous beams, also far removed from the reaction. In addition, high steel stress occurs normally at each bar cutoff point along with a local problem of stress transfer by bond. Thus, it is entirely logical to expect different shear strengths under these entirely different and more severe conditions.

INVESTIGATION PROGRAM

Scope

The investigation here reported is not the result of a carefully planned program, but rather represents a series of probing or exploratory tests to investigate the effect of bar arrangement on bond and shear strength. Several sizes and lengths of beams were used with a wide variety of steel arrangements. Most of these beams were tested on simple spans (Fig. 1) but some were loaded to include negative moments at one end (Fig. 2).

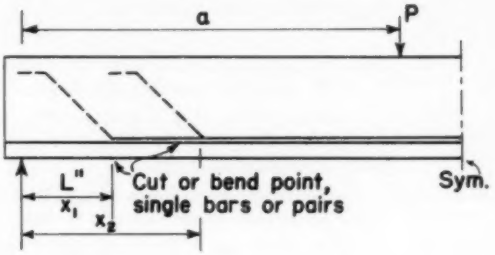

Fig. 1—Typical simple span with bars cut or bent at one or more points

A total of 55 beams are reported in Table 1. These tests have been grouped according to the number and size of their full length (straight) bars and not according to their original test series. In every case the shear span-to-depth ratio was 4.4 or more.

Materials and tests

The reinforcing steel was all hard or rail grade steel (except for Beam M29 in Group IV) with deformations satisfying ASTM A-305. A few plain bars were used in Beams M15, M16, and M22 of Group VI.

The concrete used represented several different mixes made from high-early-strength cement and Colorado River sand and gravel, the latter having a maximum size of from ¼ to ½ in. The concrete was machine mixed and vibrated into place in plywood beam forms and steel cylinder molds.

One day after casting, the specimens and cylinders were removed from the forms and placed in a moist room, where they were left for 6 days with temperature and humidity controlled. Specimens and cylinders were then removed from the moist room and surface dried. The beams were whitewashed and on the eighth day* were loaded by increments in a screw-type testing machine. Each test included deflection readings (not reported here) and the detailed marking of cracks as they developed. The test time usually ran between 1 and 2 hr.

Data

The first columns of Table 1 list the beam properties and the ultimate shear capacity V_{ult}, with calculated stresses, general notes, and type of failure also given in the later columns. The dimensions listed include the shear span a and the distances x_1 ($= L''$) and x_2 from the reaction to the bend or stop point of the bars, as shown in Fig. 1. The symbol "Bt" after a bar listing means that bar is bent up into the top of beam. All beams through Group VII were tested on simple spans except Beams M18 and M21 in Groups III, these being "continuous" type beams without stirrups, as in Fig. 2. Group VIII represents a "continuous" type beam as in Fig. 2 with stirrups added outside the L'' length in an attempt to restrict diagonal tension failure.

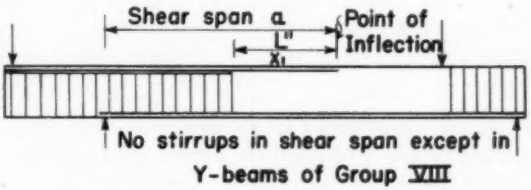

Fig. 2—Simple span beam with overhanging end, simulating continuous beam condition

*Except that Beam M10 in Group V was tested after several additional weeks in the open air of the laboratory.

Failures have been classified as follows:

Flex.	= Yielding of steel followed by secondary compression failure
Flex. + DT	= Yielding of steel followed by diagonal tension failure before reaching the secondary compression failure
DT-u	= Failure appearing as a diagonal tension crack from last bar cutoff point, with or without noticeable longitudinal cracking of concrete along bars prior to failure
DT-u-x_2	= Failure appearing as a diagonal tension crack, but occurring at a point other than where the last bar was stopped.
Split	= Splitting along the bar in the L'' length which extends far enough to free the bar from the concrete. (A diagonal crack sometimes formed in the beam thus left without reinforcing.)

Beam behavior prior to failure

With such a variety of beams, the crack patterns varied considerably. Where bars were cut off, moment cracks started at the discontinuity under a somewhat smaller load than would otherwise have been expected; that is, with cracks opening at rather regular spacings under increasing loading, the crack at the cutoff point would frequently appear and leave a greater than average spacing behind it.

Furthermore, either immediately on opening as a moment crack or soon thereafter, this crack tended to bend over and appear as the starting of a diagonal crack. Such a crack might progress over several load increments, and finally result in failure; or after several load increments, a somewhat separate diagonal crack might form and result in immediate collapse. Either of these developments would be normal in diagonal tension specimens. However, the early formation of the inclined crack might be accurately described as a premature development which seems to be associated with the bar cutoff point.

When the last cutoff point was close to the support, the cracking moment at bar cutoff was not developed until higher shears also existed. In such cases final failure in diagonal tension seemed to be coincident with the development of this moment crack at the last cutoff point.

In a few cases, marked in Table 1 as DT-u-x_2, the failure crack was not at the last cutoff point. Such a failure occurred when a single bar was stopped quite close to the support, say within $0.6d$ of the support, but there were also other cases. This variation is discussed near the end of the paper.

In many cases longitudinal splitting cracks showed over bars, indicating serious bond situations. These were more generally over the continuing bars rather than over the bars that were cut off or bent up, although sometimes both types showed this weakness.

It was not possible to establish by inspection whether failures were caused primarily by weakness in bond or weakness in diagonal tension. Some diagonal cracks stripped the bars from the concrete entirely to the end of the beam, but other cracks definitely broke out short of the support and

there was no splitting or other evidence of any bond disturbance at the end of the beam.

The behavior of the Y-beams in the last group will be discussed later along with the reasons for this type of specimen.

CALCULATION OF STRESSES

The steel stress calculations given in Table 1 were based on the moment at the vertical section and on a nominal value of j. Generally, f_s was calculated at midspan when flexure failure was involved; otherwise it was calculated at the cutoff point nearest the reaction or at failure section. These stresses were well below the steel yield point and this could be verified from the deflection curves, which failed to show the break point which yielding would produce. Where calculated stresses were large, consideration was also given to the increased effective moment which results from diagonal cracking, but such values were not tabulated. The authors believe that no yielding occurred except in those specimens marked as failing in flexure or in flexure and diagonal tension.

The maximum bond and shear stresses were calculated at the point of minimum steel from $u = V/(\Sigma ojd)$ and $v = V/(bjd)$, with j taken as 0.875. These stresses are awkward to compare directly since f_c' varied from 2470 to 4770 psi, with one specimen at 6200 psi. Since 87 percent of the specimens had f_c' within 20 percent of 3700 psi, it was decided to convert the calculated stresses into the equivalent stresses which would have existed if all beams had been made of 3700-psi concrete.

Several different relationships have been proposed to correlate ultimate v and f_c'. In an earlier paper,* it was found that $v = 14\sqrt[3]{f_c'}$ seemed to fit University of Texas data very well. (Obviously, this expression should also vary with the percent of steel, but just how much is not yet well established.) Corrections of v to v_{3700} were tried on the assumption that v varied as $\sqrt[3]{f_c'}$ and again on the assumption that v varied as $\sqrt[2]{f_c'}$. The differences were quite small and either might well have been used. In one or two places the results using $\sqrt[3]{f_c'}$ seemed to fit into a more logical pattern. Hence the cube root ratio was used.

The variation of ultimate u with f_c' is less certain. Since there were some advantages in data presentation when the same correction was used for both u and v, the calculation of u_{3700} was likewise made on the assumption that u varied as $\sqrt[2]{f_c'}$.

No subscripts are shown on v and u outside of Table 1. All curves and discussion relate to v_{3700} and u_{3700}, however.

*Ferguson, Phil M., and Thompson, J. Neils, "Diagonal Tension in T-Beams Without Stirrups," ACI JOURNAL, V. 24, No. 7, Mar. 1953 (*Proceedings* V. 49), pp. 665-676.

TABLE 1—DATA AND CALCULATIONS

Beam No.	b, in.	d, in.	a, in.	Part length bars				f_y, ksi	f'_c, psi	V_{ult}, kips	Calculated					Type of failure	Comments
				Number and size	x_1, in.	Number and size	x_2, in.				u, psi	v, psi	f_s, ksi	u_{2700}	v_{2700}		
Group I: Two #4 full length bars, 0.75 in. clear cover																	
S3	10	5	22.5	1-#4Bt	5.25	1-#4Bt	10.87	66.0	3150	10.25	745	234	65.7	785	247	Flex. + DT	Bars bent up
S1	10	5	22.5	1-#4Bt	11.25	1-#4Bt	16.50	70.2	3120	9.88	720	226	63.5	763	239	DT-u	Bars bent up
S2	10	5	22.5	2-#4Bt	11.25	—	—	59.0	3210	9.40	685	215	60.4	(715)	(225)	Flex.	Bars bent up
W3	10	5	22.5	1-#4	3.00	2-#4	10.50	—	4770	8.20	597	187	32.8	550	172	DT-u-x_2	Failure across three bars
M2	10	5	22.5	2-#4	5.25	—	—	59.0	4150	7.90	575	181	23.8	555	174	DT-u	Short bars welded to long
M8	10	5	22.5	2-#4	5.25	—	—	67.0	4300	7.70	560	176	23.2	532	167	DT-u	Short bars anchored to plate
M3	10	5	22.5	1-#4	5.25	1-#4	10.87	56.6	4420	7.65	558	175	23.0	525	165	DT-u	Welds as in M2
W4	10	5	22.5	1-#4	9.00	1-#4	13.50	—	4650	7.55	550	173	38.8	510	160	DT-u	All short bars hooked
						1-#4	18.00										
M1	10	5	22.5	2-#4	5.25	—	—	57.7	3850	7.55	550	173	23.7	542	170	DT-u-x_2	Forked ends (Fig. 6). Failure across four bars
S4	10	5	22.5	1-#4Bt	5.25	1-#4	10.87	—	3610	7.50	548	172	22.5	553	174	DT-u	One bar bent
M4	10	5	22.5	1-#4	5.25	1-#4	10.87	54.6	4240	7.40	538	169	22.2	514	161	DT-u-x_2	Failure across four bars. Forked ends (Fig. 6)
M9	10	5	22.5	2-#4	5.25	—	—	68.0	4650	7.30	532	167	22.0	492	155	DT-u	
M4a	10	5	22.5	1-#4	5.25	1-#4	10.87	—	3990	6.60	480	151	19.9	468	147	DT-u	
M1a	10	5	22.5	1-#4	9.00	—	—	57.0	2940	6.50	458	144	33.4	495	155	DT-u	
M3a	10	5	22.5	2-#4	5.25	—	—	—	3430	6.08	443	139	18.3	455	143	DT-u	
Group II: One #4 full length bar, 0.75 in. clear cover																	
M12	10	5	22.5	1-#4	7.50	1-#4	15.0	63.0	4220	4.88	711	112	41.8	683	106	DT-u	
K4	10	5	22.5	1-#4	4.50	1-#4	12.0	54.0	4050	5.44	790	124	28.0	766	120	DT-u	
K1	10	5	22.5	1-#4	1.50	1-#4	9.0	50.0	3800	5.98	870	137	30.8	866	135	DT-u-x_2	Failure across two bars
K2	10	5	22.5	1-#4Bt	7.50	1-#4Bt	15.0	50.0	3800	6.23	910	143	53.4	(902)	(142)	Flex.	
Group III: One #4 full length bar, 0.7o in. clear cover																	
M20	7	5	28.0	1-#4	9.50	1-#4	19.0	71.0	3700	3.59	522	117	39.0	523	117	DT-u	
M19	7	5	30.0	1-#4	7.50	1-#4	15.0	64.0	3650	3.95	575	129	33.8	578	129	DT-u	
						1-#4	22.5										
M25	7	5	30.0	2-#4	7.50	1-#4	22.5	63.0	3590	4.02	584	131	34.5	589	132	DT-u	
M17	7	5	30.0	1-#4	7.50	1-#4	15.0	67.0	4330	4.12	600	135	35.3	570	128	DT-u	
						1-#4	22.5										
M14	7	5	22.5	1-#4	7.50	1-#4	15.0	62.0	3730	4.28	624	140	36.8	622	140	DT-u	
M23	7	5	30.0	1-#4Bt	7.50	1-#4Bt	15.0	61.4	3710	6.16	892	202	52.9	889	201	Split?	Fragmentation
						1-#4Bt	22.5										
M21	7	5	22.5	1-#4	7.50	1-#4	15.0	62.0	3620	4.44	646	145	38.1	649	146	DT-u	"Continuous" beam
M18	7	5	22.5	1-#4	7.50	1-#4	15.0	67.5	3720	3.66	530	119	—	Discard		DT-u	"Continuous" beam. Failure suggests torsion. Discard.

Group IV: Two #3 full length bars, 0.75 in. clear cover

M24	7	5.06	30.0	1–#4	7.50	1–#4	15.0	72.7	3730	5.32	509	172	43.0	508	171	DT-u-x_2	Failure across three bars
						1–#4	22.5	65.0									f_y = 65.0 for #4 bars
M27	7	5.06	30.0	2–#3	8.25	2–#4	16.5	71.8	3700	4.96	475	160	40.3	475	160	DT-u-x_2	Failure across four bars
																	f_y of #4 not tested
M26	7	5.06	30.0	2–#4	7.50	1–#4	22.5	71.8	3720	4.46	428	144	34.4	427	144	DT-u	
M31	7	5.06	24.75	2–#3	8.25	2–#3	16.5	74.2	3760	4.52	433	146	38.3	430	145	DT-u	1.50 in. clear cover
M30	7	5.06	24.75	2–#3	8.25	2–#3	16.5	58.2	3460	4.39	418	142	37.2	427	145	DT-u	
M28	7	5.06	24.75	2–#3	8.25	2–#3	16.5	50.2	3730	4.29	410	138	36.3	410	138	DT-u	
M29	7	5.06	30.0	2–#4	11.00	1–#4	22.5	44.5	3760	3.92	378	127	44.3	375	126	DT-u	
M32	7	5.06	22.5	2–#5	11.00	—	—	67.2	3590	3.64	348	117	41.2	353	118	DT-u	

GROUP V: Two #5 full length bars, 0.75 in. clear cover

M7	10	4.94	22.5	1–#4	3.00	2–#4	7.50	67.0	4720	9.18	543	212	26.9	500	195	DT-u-x_2	Failure across three bars
M10	10	4.94	22.5	2–#4	5.25	—	—	66.5	6200	9.10	537	210	17.9	452	177	DT-u	Tested after several weeks dry storage
M6	10	4.94	22.5	1–#4	5.25	1–#4	10.87	—	4400	8.00	473	185	15.7	447	175	DT-u	

Group VI: Two #4 full length bars, 0.75 in. clear cover

M11	7.5	5	22.5	2–#4	5.25	—	—	62.1	4550	6.17	449	188	18.2	418	175	DT-u	
M13	7.5	5	22.5	2–#4	5.25	—	—	64.5	3800	5.74	418	175	17.3	415	174	DT-u	
M15	7.5	5	22.5	2–#4	5.25	—	—	66.5	3580	5.64	411	172	16.1	416	174	DT-u	Short bars plain, f_y = 60.0
								60.0									
M16	7.5	5	22.5	2–#4	5.25	—	—	57.0	3740	5.37	392	164	16.2	391	165	DT-u	Long bars plain, f_y = 57.0
								63.5									
M22	7.5	5	22.5	2–#4	5.25	—	—	56.0	3560	4.87	354	148	14.7	359	150	DT-u	All bars plain

Group VII: Full length bars as listed under comments; 0.75 in. clear cover

M33	10	5.00	22.5	2–#4	7.5	—	—	68.1	3420	5.44	525	124	42.3	538	127	DT-u	Two #3 full length
W2	10	5.00	22.5	2–#4	7.5	—	—	—	4380	8.55	414	195	24.4	392	184	DT-u	Three #4 full length
K3	10	5.00	22.5	—	—	—	—	—	3300	6.66	323	152	57.3	(336)	(158)	Flex.	Three #4 full length
M2a	10	5.00	22.5	—	—	—	—	52.0	3650	8.45	312	195	43.4	314	196	Flex. + DT	Four #4 full length
W1	10	5.00	22.5	—	—	—	—	—	4020	9.75	284	223	50.1	277	217	DT-u	Five #4 full length

Group VIII: "Continuous" beams except Y4; one #6 full length bar except Y6; 1.5 in. clear cover

Y2	4.75	5.12	38	2–#6	12	—	—	54.0	3900	4.65	440	218	53.7	432	214	Split	
Y1	4.75	5.12	38	2–#6	18	—	—	55.6	4180	4.60	435	216	43.2	418	207	Split	
Y3	4.75	5.12	38	2–#6	24	—	—	56.8	3070	3.99	377	187	48.5	401	199	DT-u	Probably DT alone
Y5	4.75	5.12	38	2–#6	24	—	—	57.0	3450	3.90	368	183	47.4	378	187	DT-u	Probably DT alone
Y6	4.75	5.19	38	2–#6	20	—	—	62.6	3660	4.15	467	193	59.1	470	196	DT-u	Probably DT alone; one #5 full length
Y7	10	5.12	38	2–#6	24	—	—	—	2470	4.74	445	105	57.8	509	120	Split	
Y4	4.75	5.12	35.5	2–#6	18	—	—	56.8	3220	3.80	360	179	34.8	378	187	DT-u	Probably DT alone; simple span

DISCUSSION OF DETAILED TEST RESULTS

Since a discussion of the effect of cutting off bars inevitably involves a variation in the percentage of steel, it seems desirable first to discuss in some detail groups of beams in which the full length bars were kept constant.

Group I

The beams of Group I all have two #4 full length bars which represent a minimum value of $p = 0.0080$. The maximum p at midspan is 0.0160 except for three beams, W3 and W4 with 0.0200 and M1a with 0.0120. The results from these 15 beams are plotted in Fig. 3. Because the ratio of beam width to perimeter is constant for the entire group, all plotted points fall on a straight line.

The upper three points in Fig. 3 represent beams with two bars bent up. Of these, the lowest value represents a flexural failure in tension, including the final secondary failure in compression. The top value represents a yielding of the steel and a near flexural failure. The difference between these values corresponds closely to the differing values of f_y. At the upper left side of the graph is a separate point for four full length bars (Beam M2a)

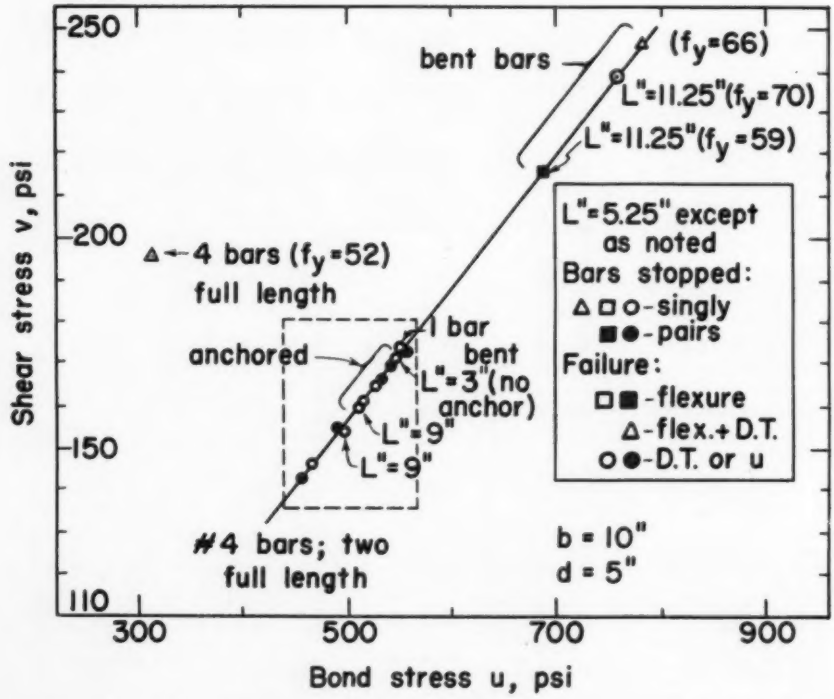

Fig. 3—Group I beams, showing effect of cutting off bars, with or without end anchorage, in comparison to bending up bars

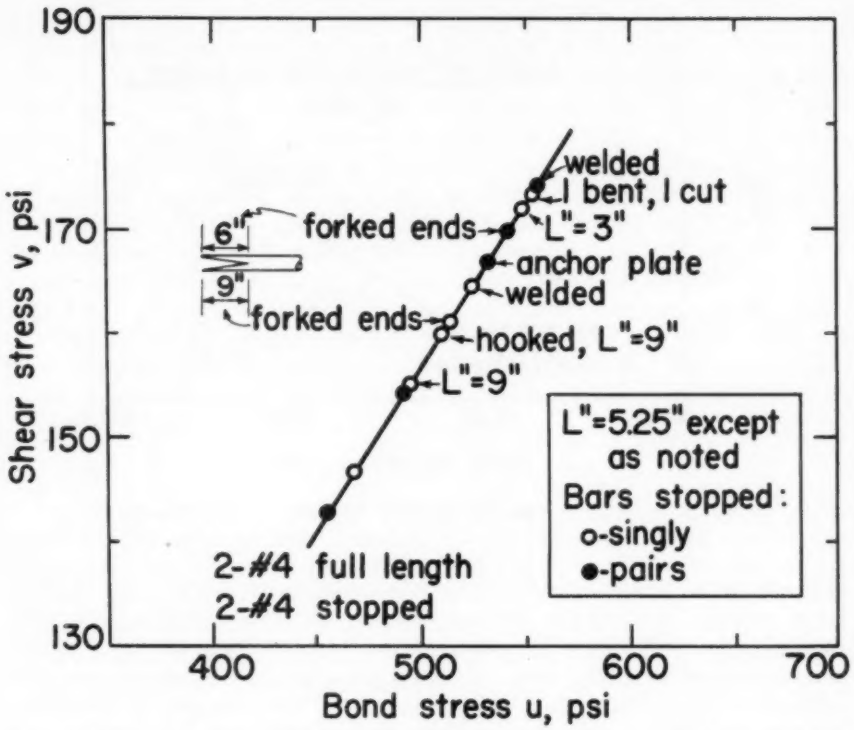

Fig. 4—Enlarged portion of Fig. 3, showing effect of various methods of anchoring the cut bars

which, for its lower f_y, agrees well with the shear strength of the other two beams. It thus appears that either full length straight bars or a combination of bent bars with full length bars will develop the full calculated moment capacity of these beams when f_y is not over 60 or 70 ksi. Even the one plotted point for a diagonal tension or bond failure shows a shear strength of 237 psi. The corresponding ultimate moment for this beam (by Whitney method) is 230 in.-kips corresponding to a V of 10.25 kips and v of 234 psi. In other words, although this beam did not fail in moment, it did support more than its calculated moment capacity.

The other 12 beams plotted in Fig. 3 all failed in diagonal tension or bond. One group, more or less anchored at cutoff points, gave shear strengths of 160 to 174 psi, substantially below the lowest value of 196 psi for Beam M2a. Another four beams, where bars were simply stopped, gave still lower shears of 143 to 155 psi.

In Fig. 4, the above 12 beams have been plotted to larger scale with notes about the types of anchorage used. In the anchored (upper) group, the hooked bars were least effective. Cutting a V-notch in the bars such as to

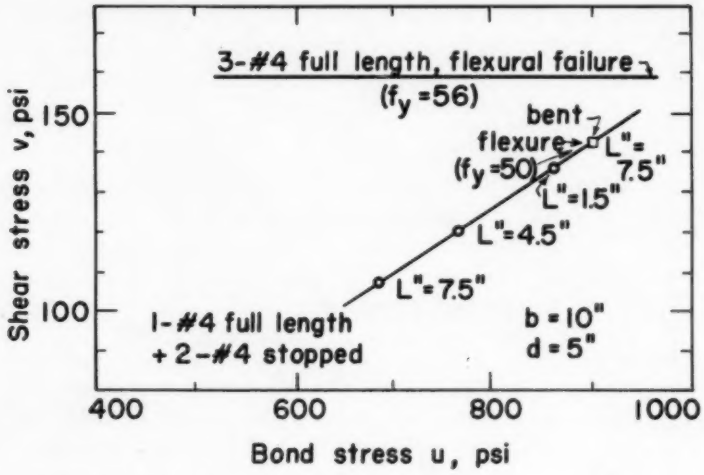

Fig. 5—Effect of varying L'' on shear strength. Group II beams

stop off metal gradually and possibly increase bond surface was slightly more helpful. Bending the cut bars laterally and welding them to the full length bars, or continuing them straight and butt welding them to an anchor plate improved strength some. It might be noted that when one bar was extended *almost* to the reaction ($L'' = 3$ in.) it was nearly as good as an anchor; likewise when the one bar closest to the support was bent up.

These anchorage devices are not practical measures, but they do raise beam strengths above those of the unanchored lower group. It thus seems apparent that a probable end slip of the unanchored bars is one of the reasons for the lower shear strengths of the unanchored lower group.

However, reference to Fig. 3 indicates that end anchorage alone does not develop the full potential shear strength available without stirrups. It appears that full-length bars or bent-up bars are essential if this full strength in shear is needed. Apparently the "sudden" transfer of tensile stress from the cut bars to the continuous bars, even when the bars are welded, adds a local longitudinal tension to the concrete which lowers the shear strength of the beam.

It is probable that the bent bars themselves help to carry some diagonal tension. Note, however, that these bars in two cases were bent up at a distance of $2.25d$ from the support. Regardless of the reasons why, the use of the bent bars in these beams increases the strength in shear by 50 percent over the detail where bars are simply stopped. It is indicated below that part of this difference can be eliminated by extending the bars that are stopped; the ACI Code requirement of an arbitrary 12-diameter extra extension is certainly in the right direction.

Group II

Group II specimens are shown in Fig. 5. Each of these beams had a minimum $p = 0.004$ with the center $p = 0.0120$. The beam with bent bars happened to have an f_y of only 50 ksi and thus failed in flexure at a fairly low shear stress. Another beam with three full length bars which would fall off the graph to the left is represented by a horizontal line. The difference in strength is in direct accord with the difference in yield point strength of the steels. The other three beams represent beams with bars stopped singly, with varying L'' distances from the last stop to the support. The ill effect of cutting off the bars increases as L'' is increased, probably because this means higher steel stress and more elongation in tension. In the case where L'' is 7.5 in. or $1.5d$, at least 25 percent of the shear capacity is lost and v is only 106 psi for a 3700-psi cylinder strength. Extending bars beyond their theoretical cutoff point reduces L'' and f_s and should be helpful in maintaining shear strength.

Group III

Another comparison between bent bars and bars simply cut off is given by Fig. 6 for Group III beams. The bent bar arrangement is from one-third to two-thirds stronger than the straight bar arrangements. The increase in L'' from 7.5 to 9.5 in. reduces strength from 8 to 16 percent. The bent bar failure is probably a splitting failure initiated at the bend points (the beam was rather narrow). Prior to failure the beam was badly cracked, with diagonal cracks, small splitting cracks, and moment cracks. It failed suddenly

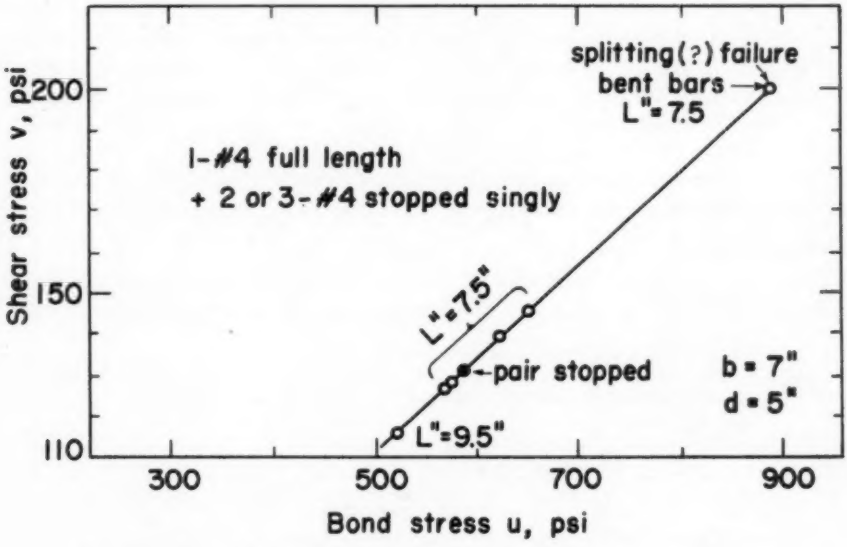

Fig. 6—Effect of varying L'' and of bending up bars. Group III beams

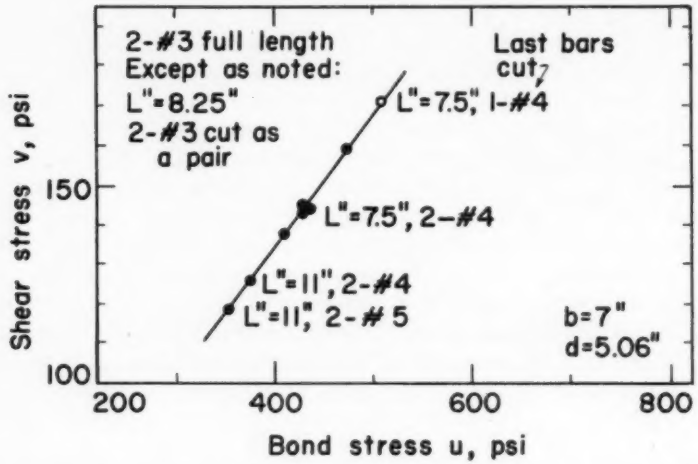

Fig. 7—Effect of varying L'' and the area of bars cut off. Group IV beams

with the concrete at one end breaking into at least a dozen fragments and leaving the bars exposed and free standing.

Group IV

The effect of stopping varied amounts of steel is shown in Fig. 7 which plots data for the beams of Group IV. These beams with a minimum $p = 0.0044$ performed a little better than those of Group III, probably because of the greater perimeter provided. The highest strength goes with the shortest L'' and a single #4 bar stopped. Stopping two #4 bars as a pair gives lower strength. A slightly longer L'' of 8.25 in. with two #3 bars stopped gives about the same values as for the two #4 bars stopped; apparently the larger L'' offsets the smaller area cut off. With L'' of 11 in. and two #4 bars stopped, the values are 18 psi lower than for $L'' = 7.5$ in., while stopping two #5 bars lowers the strength another 8 psi. It is apparent that strength decreases as L'' increases and as the area of steel cut off at one point increases.

Groups V and VI

The few specimens in Group V, plotted in Fig. 8, once again show the adverse effect of increasing L''. This figure also affords a comparison with a

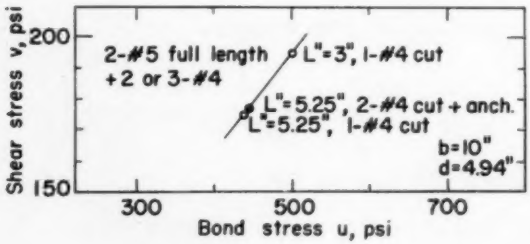

Fig. 8—Group V beams

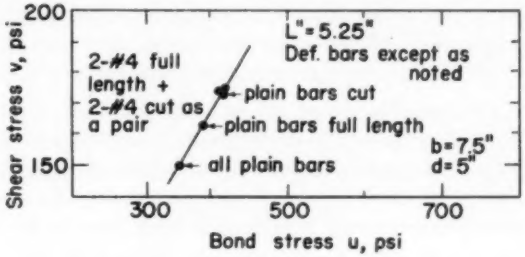

Fig. 9—Plain bars compared to deformed bars. Group VI beams

pair of bars cut off but welded to an end anchor plate. The ill effect of stopping two bars together is reduced by end anchorage and becomes no worse than that from stopping a single bar.

The effect of substituting some or all plain bars is shown in Fig. 9 which covers Group VI beams.

Groups I through VII shown together

All of Groups I through VI plus a miscellaneous Group VII have been shown together in Fig. 10. All values are corrected to correspond to $f_c' = 3700$ psi, except in the case of complete flexural failures. Since these are tension failures, the values are only slightly modified by variations in concrete

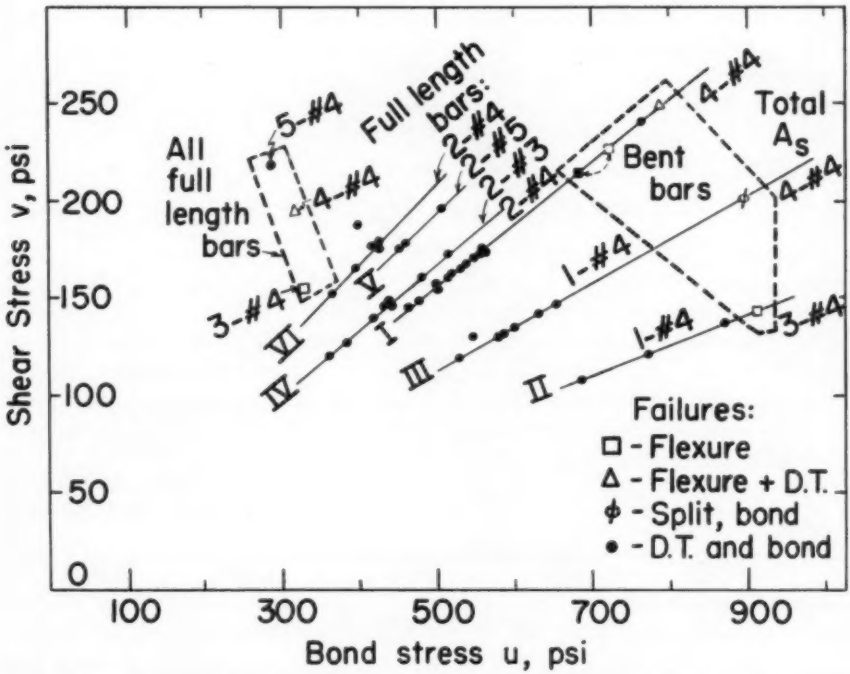

Fig. 10—Over-all comparison of full length bars, bent-up bars, and bars cut off

strength. However, in comparing shear strengths, these values converted to $f_c' = 3700$ psi may also be of interest. The converted value is shown dotted in the one case where there is any significant difference.

Attention is called to the fact that when some bars are cut off shear values generally rise with the increasing amount of full length steel. Full length bars with no cutoff give better results, as shown within the dotted area to the left. The bent bar specimens, in the dotted area at the right, also give higher strengths, entirely comparable with the results from full length bars.

INTERACTION BETWEEN DIAGONAL TENSION AND BOND

Discussion of bond stresses has been minimized in the above comparison, not because bond was unimportant but simply because it could scarcely be separated from diagonal tension. Some of the lower shear values which accompany high bond stresses are almost surely the result of bond failures. Likewise, some of the higher shear stresses accompanying low bond stresses must be more a matter of diagonal tension failure. In the majority of cases it was not possible to attribute failure definitely to one or the other cause; failure seemed to be an interaction effect. As such, the general term "shear

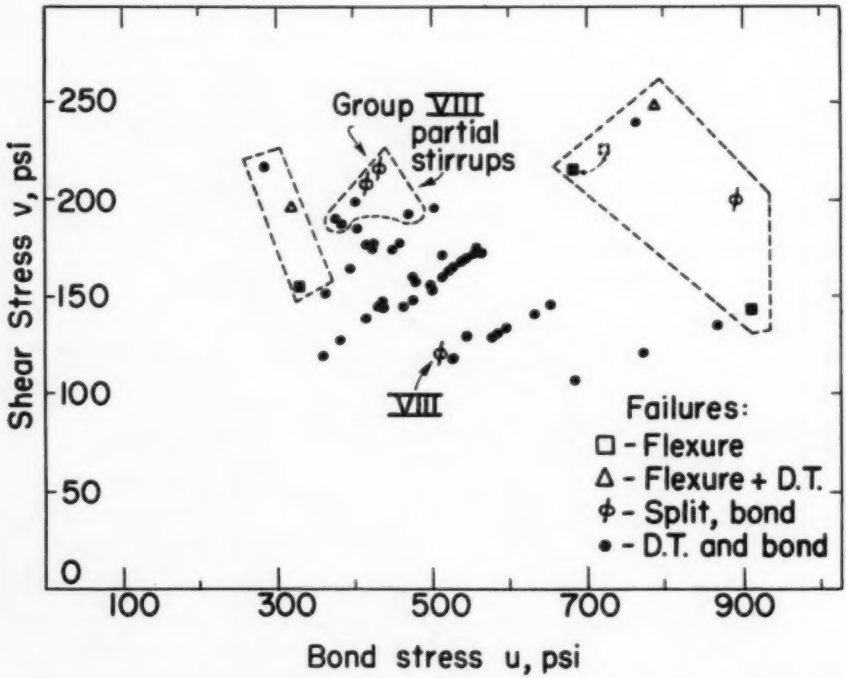

Fig. 11—The partial stirrups of Group VIII beams increase shear strength. Note that the one low value represents a splitting (bond) failure

failure" may actually be more appropriate here than when its meaning is restricted to diagonal tension failure. In most of the above discussion shear failure has been used to describe this interaction type of failure.

In some related tests it was desired to eliminate as much as possible all diagonal tension aspects and secure a specimen which would be more apt to fail in bond, probably by splitting. It was decided* to make up a series of beams of the Y-type in Fig. 2, using stirrups everywhere except in the L'' length, which in this case is measured from the point of inflection. These beams are listed as Group VIII and their strengths are superimposed in Fig. 11 on the values shown in Fig. 10. In this case, the stirrups greatly retard development of the serious diagonal cracks which tend to start from the bar cutoff points.

Three of these specimens did fail in splitting as expected, even though L'' was made much larger than in the other tests. The average bond stress developed was from 418 to 509 psi, but the failure was somewhat progressive, and undoubtedly higher unit bond stresses were developed locally during this process. It is noted that before splitting two of these beams also developed high shearing stresses. The third was a wide beam such that v remained relatively low.

In spite of the stirrups four of these beams failed in diagonal tension or shear in the general sense, beyond the stirrups and in the L'' length. The fact that these values are high compared to other beams is probably due to two factors. The heavy stirrups might be assumed to have minimized the ill effects of cutting off bars. In addition, the percentage of steel with the one rather large bar was relatively high, from 0.0126 to 0.0181.

EFFECT OF PERCENTAGE OF STEEL

The data in Fig. 10 emphasized that the percentage of steel used as full length straight bars was an important variable. In Fig. 12 the data have been replotted with this minimum p as the abscissa. There is a considerable scatter caused by various steel patterns, but in general upper and lower boundary lines can be drawn to indicate at least approximate limits. There is no reason to expect that the variation of shear strength with minimum steel percentage will be identical with that for full length steel, but the two values shown for full length steel (marked with arrows) match reasonably well. Once again, this figure shows that bent bars give better results.

THEORETICAL APPROACH

This investigation seems to indicate that two adverse factors must be considered when bars are cut and not bent up. One is the length L'' and the other is the quantity of steel cut off at a given point.

*At the suggestion of David Watstein, National Bureau of Standards, Washington, D. C.

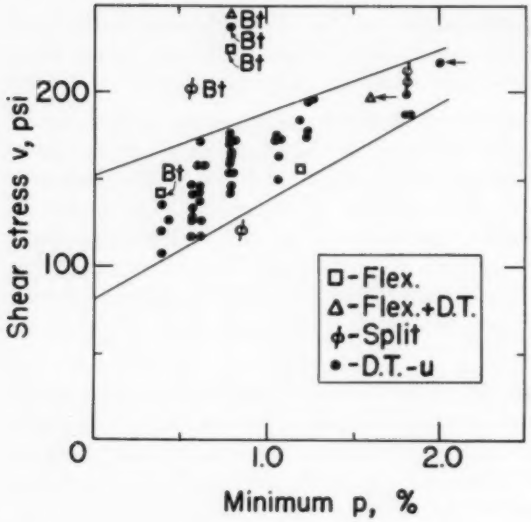

Fig. 12—Variation in shear strength as related to percentage of full length steel

One might rationalize that since cutting off bars reduces diagonal tension strength, the ill effect should be proportional to the stress these bars would carry if they continued. The effect of this transfer of tension ΔT (through the concrete to the other bars) would probably be measured by the ratio of ΔT to the concrete area, say, to bd multiplied by some constant K_1 to recognize nonuniformity of stress. Let the area of the bars cut off be designated as A_{s2} and the continuing bars as A_{s1}. Then

$$\frac{\Delta T}{K_1 bd} = \frac{f_s A_{s2}}{K_1 bd} = \frac{M}{(A_{s1} + A_{s2})\, jd}\left(\frac{A_{s2}}{K_1 bd}\right)$$

$$= \frac{Vx}{K_1 jdbd}\left[\frac{A_{s2}}{A_{s1} + A_{s2}}\right]$$

$$= \frac{vx}{K_1 d}\left[\frac{A_{s2}}{A_{s1} + A_{s2}}\right]$$

$$= Kvx\left[\frac{A_{s2}}{A_{s1} + A_{s2}}\right]$$

where $K = 1/(K_1 d)$ since d also is essentially constant for the beams tested. The total diagonal tension effect v_t might then be the ordinary v added to this extra increment.

$$v_t = v + Kvx\left[\frac{A_{s2}}{A_{s1} + A_{s2}}\right] = v\left[1 + Kx\left(\frac{A_{s2}}{A_{s1} + A_{s2}}\right)\right]$$

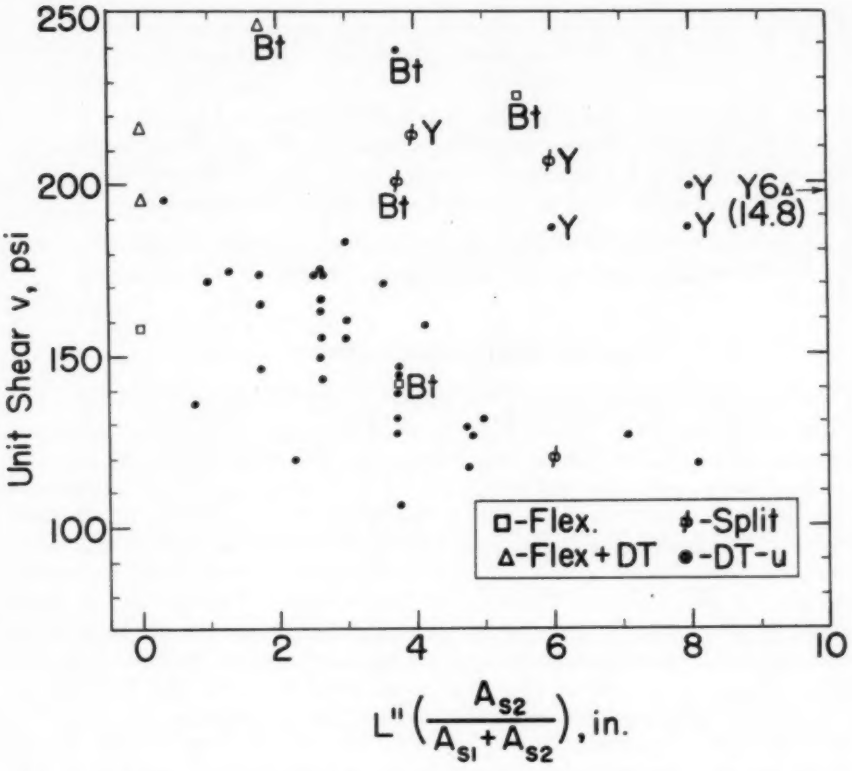

Fig. 13—Shear strength in relation to L'' and the proportion of steel cut off

Fig. 13 is a plot of the ordinary v against $L''[A_{s2}/(A_{s1} + A_{s2})]$, since generally L'' can be substituted for x. There is an obvious correlation but there is considerable scatter and one must also assume that v_t is probably not a constant, but rather a function of percentage of steel. Hence, no attempt has been made to evaluate the assumed constant K.

It is interesting that this presentation also separates the bent-bar beams quite well and shows them far above those with bars cut off. The Y-beams of Group VIII also indicate the added strength resulting from their use of stirrups at the cutoff point. Possibly the spread between the Y-beams and those plotted below correctly represents the weakness inherent because bars are cut off, but it is suspected that part of the gap simply represents the effect of the higher value of p used in the Y-beams.

The same type of relationship might be developed to express the adverse effects of bar cutoff on bond stress. In this case, the ratio of ΔT to the Σo which continues would seem to measure this effect. This leads to

$$u_i = u + K_2 ux \left(\frac{A_{s2}}{A_{s1} + A_{s2}} \right) = u \left[1 + K_2 x \left(\frac{A_{s2}}{A_{s1} + A_{s2}} \right) \right]$$

where K_2 is a new constant. When u was plotted against $L''[A_{s2}/(A_{s1} + A_{s2})]$, the points scattered so greatly in the u direction that no trend was obvious. Hence, this plot is not presented. There seems to be a little more correlation, but still inadequate, if L'' is expressed in terms of bar diameters.

It is not known whether these scattering results simply indicate that bond at the failure load was rarely important or whether the analysis itself is inadequate.

FAILURES NOT AT LAST CUTOFF POINT

Of the beams reported here, five failed at sections removed from the last cutoff point, generally about at the next cutoff point, *i.e.*, at the x_2 distance tabulated in Table 1. These beams were K1, M7, M24, M27, and W3. In each of these cases the ratio $x[A_{s2}/(A_{s1} + A_{s2})]$ was larger when calculated for the real failure section. When multiplied by the calculated diagonal tension, the product was thus increased. However, in most cases, when the factor was applied to unit bond stress the resulting value would be smaller because of the increased perimeter at this section. The average of these v and u corrections did show higher values at the inner failure section of these five beams. Unfortunately for the credence which can be placed in this comparison, it was noted that this same condition was also fulfilled by Beams K4, M4a, and M6 and these did *not* fail at the inner section. The case of Beam M4a might be explained by the increased bar tension at the outer cutoff point resulting from well developed diagonal cracking. But this explanation is not valid for Beams K4 and M6. Some further tests, not reported here, have even resulted in failures at the inner cutoff where the above comparison would have indicated greater weakness at the outer cutoff point. Further investigation is obviously required to clarify the significance of these ratios or to establish more significant ones.

CONCLUSIONS

This is a report on exploratory work and firm conclusions are therefore limited. Further investigation is definitely needed. All conclusions apply only to members without web reinforcement.

1. Cutting off bars in a tension zone will bring about some complication in web or shear stresses which will usually lower the shear strength of the member. This reduction may well be as large as one-third and may even be greater in special cases.

2. As higher strength steels are introduced and higher f_s values are used, there is some indication that the shear strength (with bars cut off) may be further reduced.

3. Bending bars instead of cutting them off will largely or completely nullify these ill results and may even increase shear strength above that for full length straight bars.

4. It is hazardous to cut off bars in a tension zone unless closely spaced stirrups are provided at the cutoff point or unless the calculated unit shear stress is quite low, say, 0.01 f_c' to 0.015 f_c'. Further investigation is needed to define more precise values and procedures.

5. Although the trend today is to eliminate bent bars, this practice seems to eliminate some sources of strength and to introduce some sources of weakness.

ACKNOWLEDGMENTS

The Bureau of Engineering Research of the University of Texas has financed a large portion of this investigation, but acknowledgment must also be made of data taken from three master's theses as follows: four W-series beams from the work of Clarence B. Williams; four K-series beams from the work of Maummer Kitapci; and four S-series beams from the thesis of Donald R. Strand.

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American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Mich.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1959, for publication in the March 1960 JOURNAL.

Design criteria, calculations, construction procedures, stress measurements, and performance history are given for two, two-way prestressed overlay slabs installed on taxiway of San Antonio International Airport. The 75 x 80-ft overlays, 4 in. thick, were placed on top of a damaged 6-in. slab. Prestress levels were 425 psi and 175 psi; both overlays were in good condition after 3½ years of service.

Prestressed Overlay Slab for San Antonio Airport

By M. M. LEMCOE and C. H. MAHLA

■ MANY AIRPORT PAVEMENTS have been damaged because of increasing gross weight of aircraft, since they were not designed to withstand the severe stresses imposed by these heavy loadings. Particularly the pavements constructed before or just after commencement of World War II show considerable damage in many places. The San Antonio International Airport is no exception. Damage is evident on both the runways and taxiways, with the taxiways having suffered most. Much of the pavement in existence at the time this investigation started was constructed before World War II or shortly after its beginning.

Southwest Research Institute was asked to investigate the merits of prestressed concrete overlays as a means of repairing Taxiway No. 1, which is subjected to the heaviest traffic occurring at the airport. To gain basic technical information on the merits of different types of overlays, plain concrete and asphaltic overlays with and without wire mesh reinforcement were also investigated. This paper, however, is limited to the prestressed concrete overlays.

DESIGN CONSIDERATIONS

It was decided to design the overlays for a DC-6 wheel loading (about 47,000 lb per dual-wheel main gear), since the predominant traffic at the airport was of the DC-6 type and performance data could be obtained sooner if the slabs were subjected to capacity traffic loads as quickly as possible.

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A cost factor also entered into the decision, since more steel prestressing tendons and metal conduits would be required for higher prestress.

The two overlays, each 75 x 80 ft and 4 in. thick, were prestressed in two directions. One slab was prestressed to 425 psi and the other to 175 psi. The two values of prestress were chosen to insure, insofar as the method of analysis allowed, that under a DC-6 loading the stronger slab (safety factor approximately 1.7 against rupture) would not fail, while the weaker (safety factor approximately 1.1 against rupture) might approach impending failure under the normal traffic load, thus providing a check on design assumptions.

The loaded structure is actually a complex system, consisting of at least four layers: the 4-in. concrete overlay, the asphalt leveling course, the 6-in. existing concrete pavement, and the underlying soil. The extensive numerical computations involved in analyzing a four layer system would, in this case, have been prohibitively expensive and time-consuming, particularly in view of the rather tenuous assumptions which would have been made in analytical formulation of the problem.

Therefore, a somewhat different approach was used, in which the amount of load transmitted to the overlay and pavement was assumed to be proportional to their respective stiffnesses as determined from standard plate bearing tests and other considerations. Specifically, the soil base and existing 6-in. pavement were considered to act as a unit, having an effective modulus of subgrade reaction k, as calculated from Burmister's formula[1]

$$k = \frac{E_2}{1.5rF_w} \qquad (1)$$

where E_2 = Young's modulus for the soil
r = radius of bearing area
F_w = settlement factor (Fig. 4, p. 133, Reference 1)

From standard bearing plate tests made on the soil base, a black clayey gumbo with a plasticity index of 26, the modulus of subgrade reaction k was established on the basis of several measurements at various locations to be 171 lb per cu in.

Stresses were then calculated for the existing 6-in. pavement, and for a 4-in. overlay (design modulus of rupture equal to 700 psi), which was considered the minimum practical thickness for the post-tensioning system used. The required prestress was determined so as to insure minimum factors of safety of 1.1 and 1.7 in the weaker and stronger slabs, respectively. The following calculations for the DC-6A wheel load illustrate the procedure used to obtain the stresses in the overlay and pavement.

Division of load between pavement and overlay

Assume tire pressure of 115 psi
Weight per main gear = 47,300 lb
Imprint area for one tire = 23,650/115 = 206 sq in.

Equivalent radius of bearing area for the 4-in. overlay = $a_4 = \sqrt{\frac{206}{\pi}} = 8.11$ in.

Assuming a 45 deg distribution of load through the overlay to the existing 6-in. pavement $= a_6 = 8.11 + 4 = 12.11$ in.

The deflection Z of the system under the wheel load may be determined from Westergaard's formula[2]

$$Z = \frac{P}{8kl^2}\left[1 + \left[0.3665 \log_{10}\left(\frac{a}{l}\right) - 0.2174\right]\left(\frac{a}{l}\right)^2\right] \quad \ldots\ldots (2)$$

where l, the radius of relative stiffness, is determined from Westergaard's relation

$$l = \sqrt[4]{\frac{Eh^3}{12\,(1 - \mu^2)\,k}} \quad \ldots\ldots (3)$$

where E = Young's modulus for the concrete, psi
h = thickness of the overlay, in.
μ = Poisson's ratio, assumed equal to 0.15
k = modulus of subgrade reaction computed from Eq. (1) or determined directly from plate bearing tests

Letting P_4 represent the portion of the wheel load carried by the 4-in. slab and P_6 the load carried by the 6-in. slab, we have

$$P_4 + P_6 = 23{,}650 \quad \ldots\ldots (4)$$

Also, since the vertical deflection of the overlay and pavement are assumed to be equal, we have

$$Z_4 = Z_6 \quad \ldots\ldots (5)$$

From Eq. (2) we obtain

$$Z_4 = \frac{P_4}{8k_4l_4^2}\left[1 + \left[0.3665 \log_{10}\left(\frac{a_4}{l_4}\right) - 0.2174\right]\left(\frac{a_4}{l_4}\right)^2\right] \quad \ldots\ldots\ldots (6)$$

$$Z_6 = \frac{P_6}{8k_6l_6^2}\left[1 + \left[0.3665 \log_{10}\left(\frac{a_6}{l_6}\right) - 0.2174\right]\left(\frac{a_6}{l_6}\right)^2\right] \quad \ldots\ldots\ldots (7)$$

From Eq. (1) we obtain

$$k_4 = \frac{2 \times 10^3}{1.5 \times 12.11 \times 0.205} = 537 \text{ lb per cu in.}$$

NOTE: k_4 represents the effective modulus of subgrade reactions for the combined 6-in. pavement and soil. From actual plate bearing tests of the soil at several locations, average value of k_6 was found to be 171 lb per cu in., as mentioned earlier.

Using the above values for modulus of subgrade reaction, the radii of relative stiffnesses l were obtained from Eq. (3) as follows:

$$l_4 = \sqrt[4]{\frac{4.5 \times 10^6 \times 4^3}{12\,(1 - 0.0225)\,537}} = 14.63 \text{ in.}$$

$$l_6 = \sqrt[4]{\frac{4.0 \times 10^6 \times 5^3}{12\,(1 - 0.0225)\,171}} = 22.4 \text{ in.}$$

The partially cracked 6-in. slab was assumed equivalent to an uncracked slab of 5 in. effective depth.

Inserting the above numerical values for k, a, and l in Eq. (4) through (7), and solving simultaneously, we obtain

$P_4 = 13{,}550$ lb = load carried by 4-in. overlay
$P_6 = 10{,}100$ lb = load carried by nominal 6-in. pavement

Maximum stress and required prestressing

Using the Portland Cement Association procedure recommended for calculating the tire imprint shape,[3] the major and minor axes of the tire imprint were found to be 9.92 in. and 5.95 in., respectively (based on previously mentioned tire pressure of 115 psi and single tire imprint area of 206 sq in.).

Maximum tensile stress at the bottom of the 4-in. overlay under the DC-6A dual-wheel loading, determined by means of influence charts prepared by Pickett *et al.*,[4] was found to be approximately 590 psi, assuming that 13,550 lb for each wheel would be carried by the overlay. Since the overlay slabs were a part of an existing taxiway, it was felt that they should be designed to accommodate the dynamic loads associated with taxiing. Lacking reliable load factor data, a load factor of 1.25 was assumed, making the dynamic tensile stress, $1.25 \times 590 = 738$ psi.

Assuming a maximum coefficient of friction between the building paper (see construction details) and the bottom of the overlay to be 1.2, the maximum loss in prestress at midlength of each slab was calculated to be 50 psi. This value for the coefficient of friction was based on measured coefficients of friction obtained by the U. S. Army Corps of Engineers, Rigid Pavement Laboratory, Cincinnati, and measured data obtained from small slabs at Southwest Research Institute. An additional 15 percent loss in prestress was assumed to account for losses due to creep in the steel and plastic flow in the concrete. On this basis, it was found that initial values of prestress of 175 psi and 425 psi would be required in the weaker and stronger slabs, respectively, to attain the desired factors of safety.

The maximum tensile stress at the bottom of the 6-in. pavement, found in a similar manner using the influence charts, was approximately 310 psi, under its share of the load P_6 = 10,100 lb per wheel. Flexural tests of beam specimens taken from three representative locations in the existing 6-in. pavement indicated an average modulus of rupture of the concrete of 647 psi. Again assuming a load or impact factor of 1.25, a stress of 388 psi under dynamic conditions was obtained. This resulted in a factor of safety of 647/388 or 1.7 for the underlying 6-in. pavement.

CONSTRUCTION

Except for some delay during the prestressing phase for the recording of stress data, the entire operation proceeded with a minimum loss of time and no unusual problems. After setting the grade stakes, a 1-in. stiff asphalt leveling course was spread and rolled to take care of unevenness in the existing 6-in. pavement. Edge forms were then erected for concreting the slabs in three lanes. To insure a minimum of friction, building paper was placed between the overlays and the asphalt leveling course. Next, the prestressing conduit and other accessories were set in place followed by the placing of the mild steel edge and corner reinforcing. Mild steel was placed along the edges to provide for increased slab stress resulting from wheel loadings at a corner or along an edge. This was considered necessary in the absence of thickened edges or the use of dowels to transfer edge loads to adjacent slabs. The mild steel also provided for possible "underprestressed" near-edge locations between the bearing plates of the end anchorages. Fig. 1 shows slab details including placement of the mild and high tensile steel, anchorages, and leveling course.

The construction joints, vertical in section and placed lengthwise of the taxiways, were unkeyed and undoweled except for the effect of the prestressing conduits which were continuous from lane to lane. To maintain the conduits in perfect alignment during concreting, the tubes, actually $\frac{3}{4}$ in. nominal diameter heavy duty electrical conduit, were held firmly in place by the edge forms and supported by chairs at frequent intervals. The edge bearing plates were held in position by spot welding them to the steel edge forms.

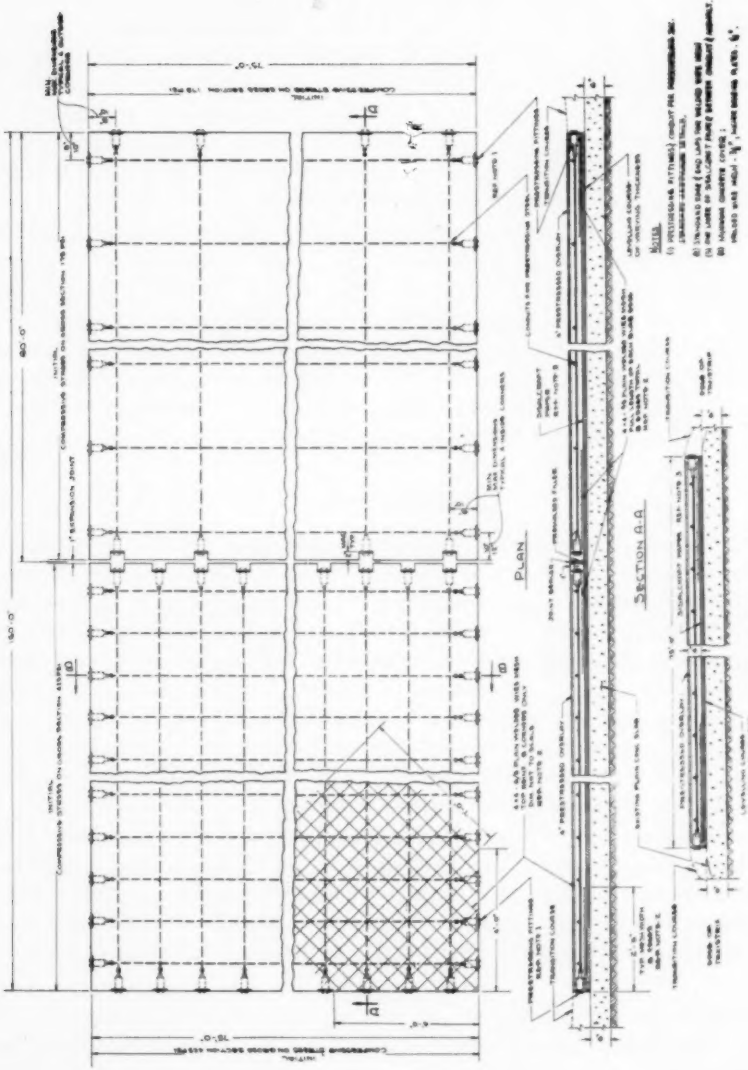

Fig. 1—Slab details showing mild and high tensile steel, anchorages, leveling course, etc.

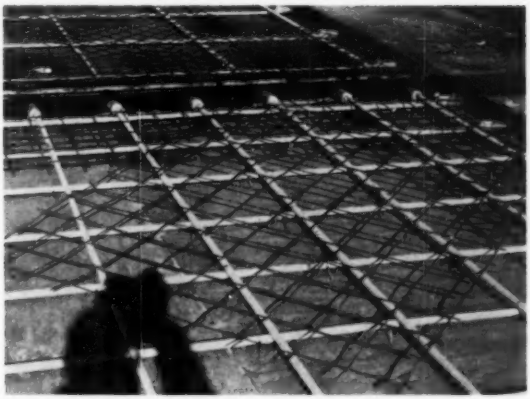

Fig. 2—Arrangement of conduit, mild steel, anchorages, and edge bearing plates before placing concrete

Later, the weld was broken by a sharp blow with a hammer to remove the forms. Fig. 2 shows the over-all arrangement of conduit, mild steel, etc., before placing concrete.

It was initially proposed to use flexible metal hose for the prestressing ducts, but this idea was abandoned because of the difficulty of maintaining alignment and eliminating sag.

Curing

The two outside lanes were completed during the first placement. Several weeks later the center and last lane was cast. Unfortunately, during the first concreting, proper curing procedures were not observed and the slab surface dried much too fast, causing shrinkage cracks to form over each conduit. Membrane type curing was specified, but due to difficulties encountered during construction, the compound was not sprayed on until the slabs had cracked and were dry enough to walk on.

For the second placement, extra spraying equipment and an overhead bridge were available. As an additional precaution, canvas was spread over the freshly sprayed concrete to keep off the drying wind. No curing cracks appeared in the second placement.

Fig. 3—Tensioning process under way

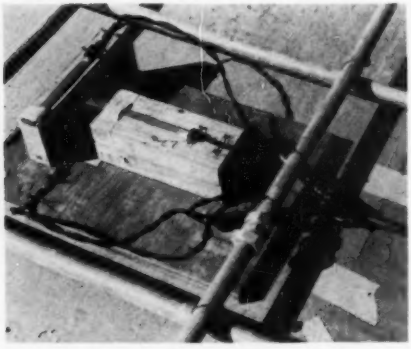

Fig. 4—Arrangement of electrical resistance strain gages before concreting

Prestressing

After the concrete had gained sufficient strength, prestressing of the slabs was begun. Four ¼-in. diameter high strength solid steel wires were inserted into each conduit and the required end anchorage hardware attached. Then, using hydraulic jacks, the four wires were tensioned as a group and shimmed at a predetermined stress. Fig. 3 shows the tensioning process in operation. When all of the steel had been tensioned, a thin grout was forced into the end of each conduit, thus protecting the prestressing steel from rust and possible release of tension due to slippage at one of the end anchors. Construction was completed and the taxiway placed in service in March, 1955.

EXPERIMENTAL CHECK OF ANALYSIS

To investigate the validity of the assumptions made in the stress analysis of the overlay system, particularly the validity of the procedure used to calculate the portion of the loads carried by the overlay, measurements were made of the tensile stresses in the two principal directions on the underside of the overlay, directly under a C-46 wheel load. (A DC-6 aircraft was not available.)

Two Valore electric resistance strain gages were installed at the center (in plan) of the overlay. The gages were placed in the wet concrete, and were carefully oriented at right angles to each other and perpendicular to the axis of loading. Fig. 4 shows the arrangement of the gages prior to concreting. The leads from these gages were brought out of the slab to the electronic instrumentation by passing them through copper conduit embedded in concrete. Dummy gages were provided on the underside of a "dummy slab," which was cast adjacent to the slab proper.

Strains and stresses under C-46 load

Strains were measured directly beneath a 12,850-lb C-46 singlewheel load having a tire pressure of 68 psi. One set of readings was taken with the aircraft pointing in the longitudinal direction of the taxiway, and a second set of readings was taken with the wheel over the same point but oriented at right angles to the first direction. The strain data obtained were as follows:

Direction of aircraft	Longitudinal gage reading, microin. per in.	Transverse gage reading, microin. per in.
Longitudinal	51	70
Transverse	70	55

Since the gages were mounted in recesses, with no normal pressure acting on the gages, a state of plane stress was assumed at this location on the underside of the overlay. The stresses were therefore computed using Hooke's equations for plane stress as follows:

$$\sigma_{long.} = E \frac{(\epsilon_{long.} + \mu \epsilon_{trans.})}{1 - \mu^2}$$

$$\sigma_{trans.} = E \frac{(\epsilon_{trans.} + \mu \epsilon_{long.})}{1 - \mu^2}$$

From tests on concrete cylinders made during placing operations in the vicinity of the gages, the modulus of elasticity of the concrete was found to be 4.22×10^6 psi. A value for Poisson's ratio of 0.15 was assumed. Substituting these values and the measured strains in the above stress-strain equations, the following stresses were obtained:

Direction of aircraft	Longitudinal stress, psi	Transverse stress, psi
Longitudinal	266	335
Transverse	338	282

On examination of the measured stresses, it is evident that the maximum and minimum principal stresses in the longitudinal direction are almost identical with the corresponding ones at right angles. Theoretically, of course, they should be equal.

Theoretical analysis for C-46 load

To check the validity of the assumptions made in the design stress analysis of the overlay system for the DC-6 loading, a similar theoretical analysis for the C-46 wheel load was conducted. It was found that the equivalent modulus of subgrade reaction was 566 lb per cu in. and that the portion of the 12,850-lb wheel load carried by the overlay was 7300 lb.

Westergaard's equation for the determination of the principal tensile stresses for the equivalent system under this loading[5] is:

$$\left.\begin{matrix}\sigma_x \\ \sigma_y\end{matrix}\right\} = \frac{P}{h^2}\left[0.275\,(1 + \mu)\log_{10}\frac{Eh^3}{k\left(\frac{a+b}{2}\right)^4} \doteq 0.239\,(1 - \mu)\frac{a-b}{a+b}\right]$$

Inserting the appropriate numerical values, $a = 9.51$ in., $b = 5.70$ in., $k = 566$, $\mu = 0.15$, $E_{avg} = 4.5 \times 10^6$, $h = 4$ in., $P = 7300$ lb, in this equation, we obtain

$\sigma_x = 292$ psi
$\sigma_y = 337$ psi

NOTE: The major and minor axes of the bearing area, a and b, respectively, were obtained using PCA method (see Reference 3.)

Note that the maximum *computed* principal stress agrees with the average *measured* maximum principal stress of 337 psi within less than 1 percent. It is felt that this provides a reasonably good check on the validity of the method which was used to reduce what in reality was a complex multilayer system down to an equivalent simple system which could then be analyzed, using Westergaard's theory for the simpler case of a slab resting on a dense liquid subgrade.

CONCLUSIONS

1. The method of analysis used in designing the prestressed overlays appears to be quite satisfactory, in view of the excellent agreement between calculated and measured maximum tensile stress.

2. Observations to date (about 3½ years of service by the slabs) indicate satisfactory performance of both the weaker and stronger prestressed overlays. Neither slab has shown any signs of distress. It was expected that the weaker slab, designed with a safety factor of 1.1, would have shown some distress by this date, since the traffic on Taxiway No. 1 has, in the past year or so, included an increasing number of DC-7 aircraft. This may indicate that the actual loss in prestress due to friction was lower than assumed in the analysis, and more recent information suggests that a somewhat lower coefficient of friction could have been used.

An attempt during prestressing operations to measure losses in prestress due to friction as well as the losses due to creep in the steel and plastic flow in the concrete was not satisfactory. Even though it is believed that adequate procedures were used to account for the strains due to temperature, the measured strains, over a period of several hours to several days, were by no means indicative of the applied prestress, due to the unavoidable volumetric changes in the concrete. Thus it was not possible to verify the friction coefficient of 1.2 used in the design. The 15 percent allowance made for loss in prestress due to creep and plastic flow may have been too conservative, which would also account in part for the good performance of the weaker slab.

Difficulties in obtaining accurate strain gage data, because of the significant volumetric changes in the concrete which occur over long periods, presented no problem in obtaining reliable data for strains under the C-46 wheel loading, since only about 15 min were required to complete all measurements.

3. The shrinkage cracks, which developed due to improper curing in both prestressed overlays have not to date noticeably enlarged, due to the closing effect of the prestress. If thin overlays built with rigid conduit of the type used here are to be successful, shrinkage must be held to an absolute minimum to prevent cracks over the conduit. Rigid conduit should be avoided in thin overlays if possible.

4. The use of a stiff asphaltic leveling course between existing pavement and overlay appears to be a practical and economical way of providing a plane surface as a base for a prestressed overlay. Theoretically, a concrete leveling course would be better because of its higher stiffness. However, feathering it out at the thin sections and along the edges might present serious practical difficulties.

5. Performance to date indicates that a prestressed concrete overlay to repair and reinforce existing runways and taxiways at the San Antonio International Airport is technically satisfactory. Whether prestressed overlays are more economical than conventional overlays remains to be seen. A 12,000 sq ft overlay would require 296 cu yd of concrete and no steel for conventional construction 8 in. thick; the same area of prestressed overlay, built 4 in. thick, would require 148 cu yd of concrete and 6050 lb steel (excluding conduits). The initial cost of a prestressed overlay will generally be greater, but reduced maintenance may well offset the differential in initial cost.

ACKNOWLEDGMENT

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Title No. 56-6

Properties of Nuclear Shielding Concrete

By JAMES O. HENRIE

General shielding requirements are discussed with particular emphasis on fast neutron shields. Concrete materials which are effective in meeting the requirements are outlined and the basic principles involved are explained. The concrete mix, type and source of materials, and strength properties of the shields of four similar nuclear reactors are compared. Effects on concrete setting time and strength of adding boron in the form of mineral colemanite and borocalcite and the counter effects of calcium chloride are described.

■ CONCRETES OF VARIOUS TYPES have been used extensively in making structures for shielding personnel and equipment from harmful nuclear particles and rays. These concretes range from ordinary portland cement concrete to concretes containing dense metallic aggregates and special cements. Where space is not an important consideration, ordinary concrete generally produces the most economical shield.[1] Where space and access through the shield become more important, as in a hot cell or nuclear reactor, the more dense mineral ore aggregates can be justified. Refined metals, such as iron and lead, are generally used as the primary aggregate only in special shields and shield components primarily due to cost considerations. Special additives such as materials containing boron have been used in some neutron shields to improve their effectiveness, but these additives may adversely affect setting and strength properties of the concrete.

It is the purpose of this paper to describe general shielding requirements, and concrete properties and mixes which best meet these requirements.

GENERAL SHIELDING REQUIREMENTS

Nuclear rays and particles commonly shielded against are gamma rays, x-rays, alpha particles, beta particles, and neutrons. Gamma-ray and x-ray shields require mass primarily, and therefore must be made of high-density material if they are to be compact. Concrete is one of the more common gamma-ray and x-ray shielding materials. Alpha and beta emitters are dangerous contaminants and require good containment but little shielding; hence concrete is not generally used for this purpose.

Neutron shielding

Neutrons are the most difficult of the above types of radiation to shield against. They must be slowed down or thermalized and then captured in the nucleus of an atom. Hydrogen is the most effective element in slowing down neutrons over the entire energy spectrum. Carbon is also quite effective, but has a low neutron capture cross section and is not commonly used as a shielding material. Most other elements are relatively ineffective in slowing down neutrons below certain energy levels. Heavy elements are generally quite effective in slowing down fast or high energy neutrons, but have little effect on intermediate and slow or thermal neutrons. Therefore, a combination of materials is required to effectively thermalize neutrons over the entire spectrum. Capture of a neutron produces a new isotope which may be unstable and might therefore release energy in the form of gammas, betas, alphas, or even other neutrons. Some are released immediately and some at a slower controlled rate. This capture and secondary release may occur when a neutron is almost through the shield and could result in a serious leakage, particularly of secondary or captured gamma rays. The shield may become radioactive due to the production and decay of unstable isotopes, and this presents a design problem where the inner shield surfaces must be intermittently accessible to personnel.

Therefore, in general, the neutron shield must be able to (1) thermalize fast neutrons, (2) capture the thermal neutrons, (3) produce secondary particles or rays which can be shielded against relatively easily, (4) effectively shield against the secondary particles produced, and (5) minimize the build-up of long-lived activity. To thermalize fast neutrons, a shield must contain light atoms such as hydrogen (usually in the form of water) to slow down neutrons by elastic or "billiard ball" type scattering and heavier nuclei (such as iron) which absorb some of the neutron's energy by the process termed inelastic scattering as it passes through the nucleus. To effectively capture the thermal neutrons, the shield must contain atoms such as boron or cadmium having high neutron capture cross sections (a measure of their ability to absorb additional neutrons). Property 3 is best displayed when the neutrons are predominantly captured in atoms which liberate energy primarily by the release of alpha particles (n-α reactions) rather than hard gamma rays (n-γ reactions). As an example, captures in boron result in n-α reactions while captures in cadmium and iron result in n-γ reactions. Property 4 is best achieved by dense materials since the important secondary energy release requiring special shielding is in the form of gamma rays. Property 5 is achieved by materials such as boron and lithium which have high capture cross sections and produce stable or short-lived isotopes on the capture of a neutron. If Property 5 is of major importance for access reasons, certain materials having long half-life products must be limited. (In a recent design of an experimental reactor shield where inside access will be necessary after the reactor has operated, limits of 4.3 percent iron, 0.12 percent zinc, 0.029

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percent antimony, and 0.0004 percent cobalt in the concrete shield were established.)

Combined neutron and gamma-ray shielding

The nuclear reactor biological shield must be effective against both neutrons and gamma rays. The design problem becomes one of balancing the properties of the shield so that the biological effects of either the neutrons or gamma rays leaving the shield will not be grossly predominant, and so that their combined effects will be below established tolerance levels.

The problems of determining the magnitude of a neutron and gamma-ray source entering a shield and the effectiveness of various shielding materials in reducing this source—thereby providing the means to determine the required thickness—are beyond the scope of this paper. Many available text books and technical references deal with problems of this type.[2]

CONCRETE SHIELDING FOR A PARTICULAR REACTOR

After the shielding problem has been thoroughly investigated for a particular reactor, it then becomes possible to write a specification for the concrete shielding material. This specification should adequately and economically meet shielding requirements and still allow some flexibility in selection of materials even though the shield thickness is fixed. This approach is particularly desirable when reactors of a particular design are to be constructed in various parts of the world.

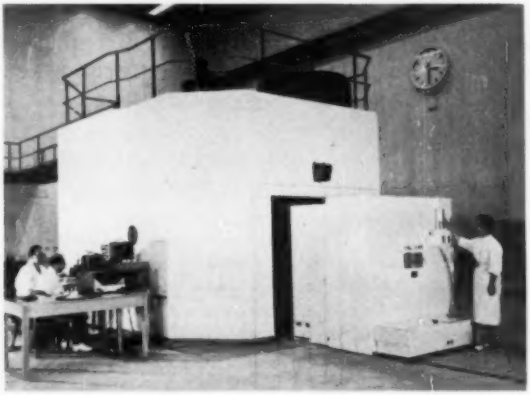

Fig. 1—A 50-kw homogeneous solution-type research reactor

TABLE 1—SHIELDING CONCRETE MATERIALS

Item	Specification requirements	Reactor location			
		Chicago	Mito	Frankfurt	Berlin
Aggregate*	Magnetite	Hematite and mill scale	Magnetite	Magnetite	Magnetite
Source	—	Mich.	Okayama Prefecture	Sweden	Africa
Specific gravity	4.6	4.4; 4.5	4.7; 4.8	4.7	4.7
Iron content, percent	64	61; 65	67	65	65
Boron additive†	Colemanite	Colemanite	Borocalcite	Borocalcite	Borocalcite
Percent boron	9.3 minimum	10	13	13.2	13.2
Specific gravity	—	2.42	2.4	2.4	2.4
Cement type	Portland Type II	Portland Type I	ASANO‡ mass concrete	Dyckerhoff‡ sulfadur Z-225	Dyckerhoff‡ sulfadur Z-225

*Iron ore aggregate graded as shown in Table 2.
†Graded to − No. 8 + No. 30 mesh. Fine borocalcite or high quality boron frit are also approved. For source of boron additive, see Table 4.
‡Equivalent to ASTM C 150 Type II portland cement.

Fifty-kw homogeneous solution-type reactors (see Fig. 1) have been installed in Chicago; Mito, Japan; Frankfurt and Berlin, Germany; and Milan, Italy. Another is planned for Washington, D. C. The same general specification was followed in establishing the dense concrete biological shield mixes for each reactor. The specification describes three basic concrete mixes to be used in various regions of the reactor shield. Mix 1 is a balanced neutron and gamma-ray shield and surrounds the core and reflector. Mix 2 serves the same purpose except that it has a slightly higher density, is limited to a $\frac{3}{4}$-in. maximum size aggregate, and is used in the special shield plugs and blocks. Mix 3 is primarily a gamma-ray shield, has a higher density than Mix 1 and is used in the subpile area. Tables 1, 2, and 3 list the properties of the materials and concrete mixes as specified and actually used in the first four reactors noted above. The concrete block in Fig. 2 is made of Mix 1.

BORON ADDITIVES IN NEUTRON SHIELDS

The primary difference between Mixes 1 and 2 and most other magnetite iron ore concrete mixes is that they contain a boron additive. The use of additives containing boron in shielding concretes has been considered debatable primarily due to their cost and deleterious effects on the setting time and strength of the concrete.[3]

Advantages must be weighed against disadvantages for each application. For the previously described 50-kw solution-type reactor, boron additives offer the following advantages:

1. The thermal column must be accessible after the reactor is shut down. Boron added to the dense concrete significantly reduces the build-up of long-lived activity. If the concrete were not borated, a boral[4] (aluminum and boron carbide mixture) liner or an equivalent boron-containing liner would be necessary.

2. Detailed shielding calculations indicate that the required shield thickness using the borated concrete of Mix 1 is 5 ft 5 in., and that 5 ft 10 in. is required for Mix 3

even though this mix has a higher density. The additional 5 in. would add 14 cu yd to the shield above floor level. The total in-place cost of this concrete is estimated at $200 per cu yd or $2800.

3. The decreased thickness also allows a slightly smaller building size, shorter shield plugs and sleeves, and higher fluxes for beam experiments at the end of open beam tubes.

Disadvantages include:

1. The additional effort required in developing and testing the borated mix.

2. The increased cost of the 85 cu yd of dense concrete above floor level. The 300 lb of colemanite per cu yd costs about $21. This replaces 570 lb of iron ore per cu yd which saves $10. The additional cement costs $0.50 per cu yd. Therefore, the net cost is about $11.50 per cu yd, or a total increased cost of about $1000.

TABLE 2—GRADING OF IRON ORE AGGREGATE

Sieve size	Aggregate, percent retained		
	Coarse	Medium	Fine
3-in.	100		
1½ -in.	95–100	100	
¾-in.	0–10	90–100	
⅜-in.	0–3	25–50	100
No. 4		0–10	90–100
No. 8		0–3	70–90
No. 16			45–75
No. 30			25–50
No. 50			10–25
No. 100			2–10
No. 200			0–5

It therefore appears advantageous to add about ½ percent boron to the dense concrete mix for this particular application.

Borated concretes would probably not prove economical for reactors surrounded by heavy steel tanks, and thermal shields. Most of the thermal neutrons are captured by the steel and result in the production of a significant secondary gamma-ray source. A properly located boron-containing thermal shield would significantly reduce this build-up and also the long-lived radioactivity. The only remaining advantage of adding boron to the concrete shield is its ability to capture the originally fast neutrons as they become thermalized and thereby reduce the build-up of secondary gamma rays.

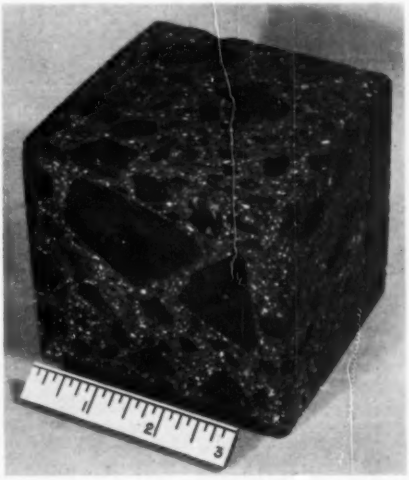

Fig. 2—Magnetite iron ore concrete containing ½ percent boron in the form of borocalcite (white particles)

Some of the more commonly used boron-containing additives (see Table 4) such as colemanite, borocalcite, and boron frit, all present to some extent the problem of increasing the setting time of the concrete.

Colemanite

Colemanite is generally considered the poorest of the three in this regard. However, it has the advantages of being more economical in some areas and having a high water of crystallization content which increases its use-

fulness in a fast neutron shield. Pure mineral colemanite is 50.9 percent B_2O_3. Good quality colemanite ore contains only about 30 to 35 percent B_2O_3. The primary impurities are gypsum and limestone. Properties of portland cement concretes containing colemanite are somewhat unpredictable and seem to depend on the chemical and physical composition and particle size of the colemanite ore used, mixing time and conditions, and temperature conditions

TABLE 3—MIX DATA AND PROPERTIES OF NUCLEAR SHIELDING CONCRETES

Item	Specification requirements	Reactor location			
		Chicago	Mito	Frankfurt	Berlin
Mix 1, used as main reactor shield					
Cement, lb per cu yd	560	564	550	574	600
Water, lb per cu yd	325*	315	270	287	350†
Aggregate, lb per cu yd					
Coarse	1615‡	2091	1550	1817	1770
Medium	1615‡	1220	1550	1817	1770
Fine	1615‡	484	2240	1817	1770
Mill scale	—	1000	—	—	—
Boron additive, lb per cu yd	300	300	240	217	230
Total weight, wet, lb per cu yd	6030 minimum	5974	6400	6529	6490
Specific gravity, wet	3.6 minimum	3.53	3.79	3.88	3.83
Slump, in.	2 maximum	1	1	½-1	1
Compressive strength, psi					
3 days	1000		1800		2300§
7 days		700**	3750	3600	3050
14 days		1512			
28 days		2069	7500		6400
Mix 2, used in shield plugs and blocks					
Cement, lb per cu yd	610	610	675	635	600
Water, lb per cu yd	350*	350	275	350	380†
Aggregate, lb per cu yd					
Medium	1450	1450	1870	1745	1600
Fine	2300	2200	2295	2620	2550
Iron scrap,††	1400	1500	1495	1460	1570
Boron additive, lb per cu yd	300	300‡‡	325	228	232
Total weight, wet, lb per cu yd	6410 minimum	6410	6935	7038	6992
Specific gravity, wet	3.8	3.8	4.1	4.16	4.1
Slump, in.	2 maximum	2	0		1
Compressive strength, psi					
3 days	1000 minimum	1000	2730		3700
28 days				4500	5700
Mix 3, used primarily as a gamma-ray shield in the subpile area					
Cement, lb per cu yd	520	634.5	570	545	558
Water, lb per cu yd	325*	325	390†	295	347†
Aggregate, lb per cu yd					
Coarse	1835‡	1381	1670	1993	1970
Medium	1835‡	936	1650	1993	1970
Fine	1835‡	1484	2250	1993	1970
Mill scale	—	1309	—	—	—
Total weight, wet, lb per cu yd	6350	6070	6530	6819	6815
Specific gravity, wet	3.77	3.6	3.87	4.0	4.0
Slump, in.	2 maximum	2½	1½	½	1
Compressive strength, psi					
3 days	1000 minimum			3890	2640
7 days		2220		4685	3850
11 days			3050		
28 days					6400

*To be trial adjusted for a 2-in. slump.
†Based on dry aggregates. Other mixes have not been corrected for aggregate surface water.
‡To be trial adjusted for optimum density.
§Samples stored at 8 C had a 3-day test strength of 1040 psi.
**Wide variation in 7-day samples.
††The iron scrap is cleaned and graded to ¾ in. maximum and ¼ in. minimum size. Pieces larger than 1 in. or smaller than ⅛ in. in any dimension or pieces having interior voids such as nuts and washers are rejectable.
‡‡One lb $CaCl_2$ was added per 94 lb cement to offset the effects of the colemanite.

during setting and curing. Therefore, trial mixes under field conditions are essential. Concrete made in accordance with Mix 1 specifications on a relatively warm day (temperature about 90 F) had a normal initial setting time of 30 to 45 min. The 80-day strength averaged 3444 psi. Using colemanite from the same source and graded to the same specification, a similar mix had a long and erratic setting time in actual field temperature conditions of about 40 F. Test cylinders at ages up to about 7 days showed low, nonuniform strength characteristics. Cylinders tested at 14 and 28 days had adequate strength and were uniform (Mix 1, Chicago reactor).

Borocalcite and boron frit

Mineral borocalcite is a calcium borate similar to colemanite, and containing 52 percent B_2O_3 in its pure form. It has been successfully used as a boron additive in three of the concretes shown in Table 3. The ore used in each shield was about 45 percent B_2O_3.

Boron frit is a refined product formulated in many different compounds. Those containing excessive sodium are not suitable for use in portland cement concretes. One of the better frits is produced by fusing boric acid, calcium carbonate, silica, and alumina and quenching the melted product in water. Increased amounts of silica and alumina raise the melting point and decrease the solubility of the end product.

Sodium impurities often found in boron-containing materials are detrimental to the setting of portland cement concrete. Gerstly borate, which contains about 5 percent sodium oxide, has been used as a boron additive in portland cement concrete trial mixes with the result that it completely inhibited setting of the concrete.

TABLE 4—BORON-CONTAINING MATERIALS SUITABLE FOR USE IN CONCRETE

Material name		Colemanite	Borocalcite	Boron frit
Chemical name and basic formula		Calcium borate $Ca_2B_6O_{11} \cdot 5H_2O$	Calcium borate $CaB_4O_7 \cdot 4H_2O$	Fused boric acid, calcium carbonate, silica, and alumina
Chemical composition* (semiquantitative)				
Boron	B	10.1	13.2	17.5
Silicon	Si	2.4	1.	14.
Aluminum	Al	0.2	0.2	7.
Calcium	Ca	24.	21.5	21.
Magnesium	Mg		0.3	0.05
Sodium	Na	0.3		0.1
Iron	Fe	0.34	0.1	0.08
Chromium	Cr	0.000		0.04
Zinc	Zn	0.014		0.015
Cobalt	Co	0.0008		0.0009
Antimony	Sb	0.0		0.00
Zirconium	Zr	0.009		0.09
Source		U. S. Borax Co., Los Angeles, Calif. Mined in Death Valley, Calif.	Borax Consolidated Ltd., London. Not generally available in U.S. Readily available in Europe-Turkey	Ferro Corp., Los Angeles, Calif., Glastex, Los Angeles, Calif. (Many others)
Approximate cost, graded, bagged, FOB source		$125 per ton	Estimated about $50 per ton at European source	$420 per ton

*This analysis is shown for general information only and cannot be considered typical of all colemanite and borocalcite ores or boron frit mixtures.

Reducing harmful effects of boron

To reduce the effect of boron additives on concrete mixes, all of the fines passing a No. 30 sieve are usually eliminated. This lessens the exposed surface area per unit weight of the material and thereby reduces its solubility. The larger particle size has the disadvantage of distributing the boron less uniformly and thereby making it less efficient in capturing neutrons. This effect is minimized by limiting the maximum size to that passing a No. 8 sieve. Where trial mixes show that a particular boron additive is not excessively detrimental, a finer grind should be used.

The Japanese made a series of concrete paste setting time tests using the Vicat needle technique. The results are shown in Table 5 and Fig. 3. The effect of colemanite on the setting time was not severe and, surprisingly, was less than for borocalcite. The addition of about 1 lb of calcium chloride per 100 lb of cement completely overcame the setting delay attributable to borocalcite. Table 6 shows strength test results for magnetite concrete containing (1) no additives, (2) colemanite, (3) borocalcite, and (4) borocalcite plus calcium chloride. The colemanite and borocalcite both reduced the strength, and the mix containing borocalcite and calcium chloride had strength comparable to the mix containing no additives.

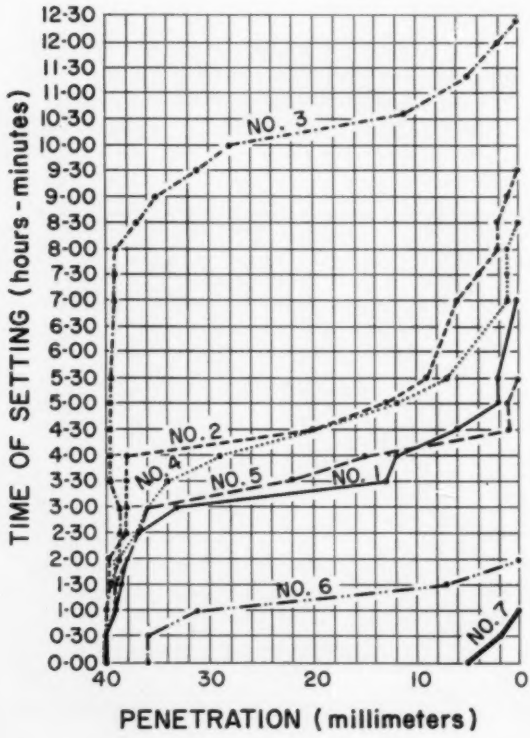

Fig. 3—Setting time of concrete pastes (see Table 5)

GENERAL PHYSICAL PROPERTIES OF DENSE CONCRETES

Structural requirements of concrete shields are usually quite low due to the tremendous area of the thick shield sections. Generally, any major loads are due to thermal stresses. As with ordinary concretes, the properties of dense concretes can be varied over a wide range to meet the particular requirements by the proper selection of materials and proportioning of the mix. Dense mixes proportioned on a volume basis can be expected to have structural properties similar to those of the same mix made with ordinary aggregates. Where a hard aggregate such as magnetite iron ore is used, the concrete strength increases.

Thermal properties such as specific heat and thermal conductivity are important where thermal stress problems are encountered. These properties are similar in magnetite concrete and ordinary concrete.[1,2] They are 25 to 50 percent lower for barite concretes, whereas the conductivity is considerably higher for concretes containing refined metallic aggregates.

Workability of dense concrete mixes is in general much poorer than that of ordinary concretes. This is due primarily to (1) the dense aggregates generally crushing into more sharp, angular, and elongated particles; (2) the tendency to decrease the amounts of air-entraining agents, cement, and free water to increase the density; and (3) the dense materials simply require more effort to move.

TABLE 5—PASTES USED FOR VICAT NEEDLE PENETRATION TESTS*

Paste number	1	2	3	4	5	6	7
Materials, weight in oz							
Cement	9.5	9.5	9.5	9.5	9.5	9.5	9.5
Water	3.32	3.32	3.32	3.32	3.32	3.32	3.32
River sand	5.55	—	—	—	—	—	—
Colemanite	—	5.55	—	—	—	—	—
Borocalcite	—	—	5.55	5.55	5.55	5.55	5.55
$CaCl_2$	—	—	—	0.050	0.087	0.202	0.304

*See Fig. 3 for setting time.

TABLE 6—COMPARISON OF MAGNETITE CONCRETE MIXES

Test number	1	2	3	4
Materials by percent of total weight				
Cement	7.8	8.1	8.1	8.1
Water	4.7	4.8	4.9	4.9
Colemanite	—	3.3	—	—
Borocalcite	—	—	3.3	3.3
$CaCl_2$	—	—	—	0.086
Magnetite iron ore				
Medium	47.2	48.7	48.6	48.6
Fine	40.3	35.1	35.1	35.0
Slump, in.	2¾	1.0	11/16	¾
Strength, psi				
1 day	980	680	680	940
7 days	3300	2830	2700	3860
28 days	5210	4490	4340	5150

CONCLUSIONS

Concrete can be designed with an effective balance of nuclear and physical properties for a wide range of shielding requirements, and becomes a flexible and economical neutron and gamma ray shielding material having excellent structural qualities. In many concrete reactor shields, boron-containing mixes are practicable and economically justifiable.

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Title No. 56-7

Width of Cracks in Concrete at the Surface of Reinforcing Steel Evaluated by Means of Tensile Bond Specimens

By DAVID WATSTEIN and ROBERT G. MATHEY

Tensile bond specimens simulating a segment of the tensile portion of a reinforced beam between two successive tensile cracks were tested to gain information on the width of crack at the surface of the reinforcing bar. Magnitude of crack width was estimated by comparing the over-all extension of the embedded bar with the extension of concrete adjacent to the bar. Crack width at the surface of a well-designed deformed bar was found to be significantly less than the width of crack measured at the exterior surface of concrete.

■ THE IMPORTANCE OF LIMITING CRACK WIDTH in reinforced concrete to safeguard the steel against corrosion is universally recognized.

In the numerous published reports on cracking of concrete, values reported for width of crack have been those observed at the exterior surface of the concrete. However, there is indirect evidence[1] that for well-designed deformed bars the crack width at the interface between steel and concrete is considerably smaller than the width at the surface of the concrete. The tensile bond specimen described in the following pages was devised to verify the indirect observations. The specimen simulates a portion of a beam between two successive cracks and was designed to furnish information on the reduced crack width at the interface as compared with that at the surface of concrete.

TENSILE BOND SPECIMENS AND MATERIALS

The test specimens (Fig. 1) were 6 x 6-in. prisms, 8 in. long, with a reinforcing bar embedded along the longitudinal axis of the specimen. Length of the specimens was determined by trial to obtain a specimen of maximum length which would not develop transverse cracks during the test.

DAVID WATSTEIN, chief of the Structural Engineering Section, National Bureau of Standards, Washington, D. C., has written several papers for the ACI Journal describing research on control of crack width and distribution of bond stresses in reinforced concrete. His April 1953 paper on the properties of concrete at high rates of loading, was awarded the Wason Medal for the Most Meritorious Paper for that year. Mr. Watstein, has been an ACI member since 1941; he now serves as chairman of Committee 208, Bond Stress, and is a member of Committee 716, High Pressure Steam Curing.

As structural research engineer in the Structural Engineering Section, National Bureau of Standards, ACI member **ROBERT G. MATHEY** has been engaged in studies of beams reinforced with high yield strength steel. He was coauthor with Mr. Watstein of an earlier Journal paper on shear tests of beams. Prior to joining the National Bureau of Standards in 1955, he was bridge engineer in the Washington, D. C., Highway Department.

The tensile bond specimens were tested by applying a tensile force to the ends of the reinforcing bar. Measurements during the test included determinations of the over-all extension of concrete at points $\frac{3}{8}$ in. from the surface of the bar, the extension of the embedded length of the bar, and the strain distribution on the sides of the specimens. It was impossible to make all of these measurements simultaneously on a given specimen with the instrumentation as shown in Fig. 1. Accordingly, data on various properties of the bond specimens were obtained from a series of tests in which a single specimen yielded information only on a single property. Table 1 gives the schedule of specimens and describes the tests performed on them.

The first letter of the specimen designation indicates the kind of measurement made—extension of steel (S), extension of concrete (C), and distribution of strain (D). The second letter identifies the pattern of bar deformations (Types A, B, and C); Type D was a smooth round bar (see Fig. 2). The last number in the designation distinguishes individual specimens within a given group.

Materials

Mix proportions of the concrete in the bond specimens were 1:2.7:3.8, by weight, and the water-cement ratio was 0.6. The aggregates were siliceous sand and gravel, and the maximum size of the coarse aggregate was $\frac{1}{2}$ in. Most of the specimens were made with a portland cement specified for general concrete construction (ASTM Type I). However, in order to expedite the work, several specimens were made with high-early-strength cement (ASTM Type III). Although the age of the specimens varied considerably, they were all moist cured until test. The compressive strength of the concrete (Table 1) ranged from 3750 to 5850 psi, as determined with 6 x 12-in. cylinders.

Properties of the reinforcing bars are also given in Table 1, and stress-strain curves are given for all deformed bars of Type A in Fig. 3. Stress-strain relationships for the other bar types were not determined.

TABLE 1—SCHEDULE OF TENSILE BOND SPECIMENS AND PROPERTIES OF MATERIALS

Designation of specimen	Measurement during test	Diameter of bar, in.	Surface characteristic of bar	Compressive strength of concrete, psi	Yield strength of reinforcement, psi	Modulus of elasticity of steel, psi
S-A7-1	Extension of steel	0.875	Deformed, Type A	4900	104,000*	28.67×10^6
S-A7-2	Extension of steel	0.875				
S-A6-1	Extension of steel	0.750	Deformed, Type A	4780	99,000*	28.54×10^6
S-A6-2	Extension of steel	0.750				
S-A5-1	Extension of steel	0.625	Deformed, Type A	4650	103,000*	29.35×10^6
S-A5-2	Extension of steel	0.625				
S-A4-1	Extension of steel	0.500	Deformed, Type A	4460	114,000*	28.99×10^6
S-A4-2	Extension of steel	0.500				
C-A-1	Extension of concrete	0.875	Deformed, Type A	5570	38,000†	29.2×10^6
C-A-2	Extension of concrete	0.875				
C-B-1	Extension of concrete	0.875	Deformed, Type B	3750	44,000†	—
C-B-2	Extension of concrete	0.875				—
C-C-1	Extension of concrete	0.875	Deformed, Type C	5550	40,000†	—
C-D-1	Extension of concrete	0.875	Smooth round, Type D	5850	55,000*	—
C-D-2	Extension of concrete					
C-A-3	Extension of concrete	0.875	Deformed, Type A	5720	69,000†	28.8×10^6
C-A-4	Extension of concrete	0.875				
C-A-5	Extension of concrete	0.875	Deformed, Type A	4140	104,000*	28.7×10^6
C-A-6	Extension of concrete	0.875				
C-A-7	Extension of concrete	0.875	Deformed, Type A	4140	104,000*	28.7×10^6
C-A-8	Extension of concrete	0.875				
D-A-1	Distribution of strain on concrete surface	0.875	Deformed, Type A	5290	104,000*	28.7×10^6
D-A-2	Distribution of strain on concrete surface	0.875				

*Yield strength determined by 0.2 percent "offset" method.
†Yield strength determined by "halt of the gage" method.

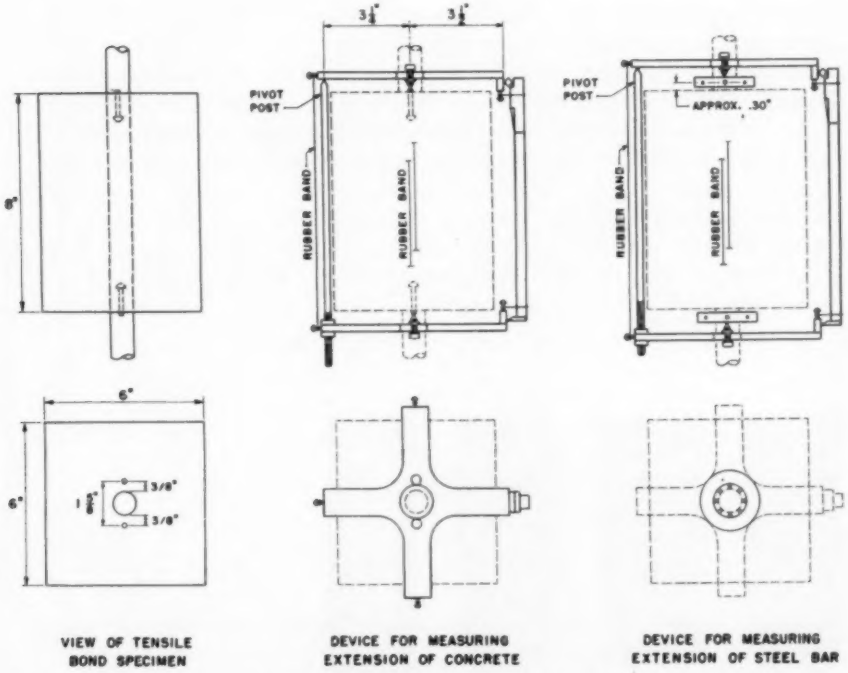


Fig. 1—Views of tensile bond specimen and instrumentation used to measure extensions of concrete and embedded bar

TEST RESULTS

Extension of concrete near the bar

The extension of concrete was determined in 13 specimens at points 0.375 in. from the surface of the bar as shown in Fig. 1.

The effect of the surface characteristics of the reinforcing bars on the extension of concrete is illustrated in Fig. 4. The performance of Bars A and B (deformed) was nearly the same, and Bar C showed the highest bond value as indicated by the extension of concrete. The smooth round Bar D produced only a negligible extension in concrete. The large difference in extensions produced with deformed and plain bars confirms similar results reported by Watstein and Seese.[2]

The relation between stress at the ends of the embedded reinforcing bar and extension of the surrounding concrete for bars of a given type is illustrated in Fig. 5.

Extension for embedded bars

Extension of the embedded bars was determined in Series S of eight specimens reinforced with bars of Type A as shown in Table 1.

Differences in extension for embedded bars of different diameters are illustrated in Fig. 6 where the ratio of average strains for the free and embedded bars is plotted against stress at the ends of the embedded bar (applied stress). This ratio is represented in the graph by the numerically equal ratio of the "effective" modulus of the embedded bar to that of the free bar. The "effective" modulus of the embedded bar was taken as the ratio of applied stress to average strain in the embedded length. At an applied stress of 20,000 psi, the ratio E_e/E_f varied between 1.55 and 1.60. The effect of bar diameter is brought out clearly at the higher values of stress. At an applied stress of 90,000 psi, the ratio E_e/E_f ranged from 1.06 for a $\frac{7}{8}$-in. bar to 1.21 for a $\frac{1}{2}$-in. bar.

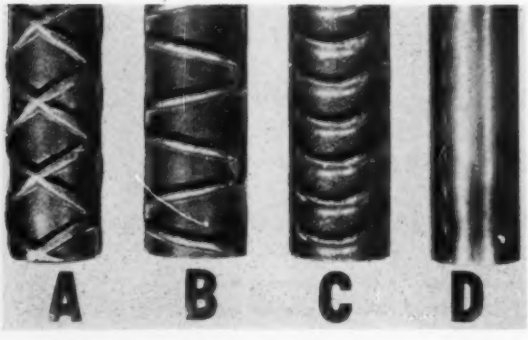
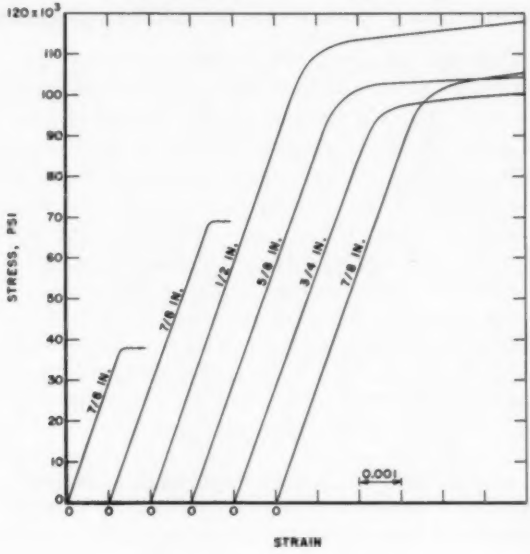


Fig. 2—View of reinforcing bars used in bond specimens

Fig. 3—Stress-strain relationships for deformed bars of Type A



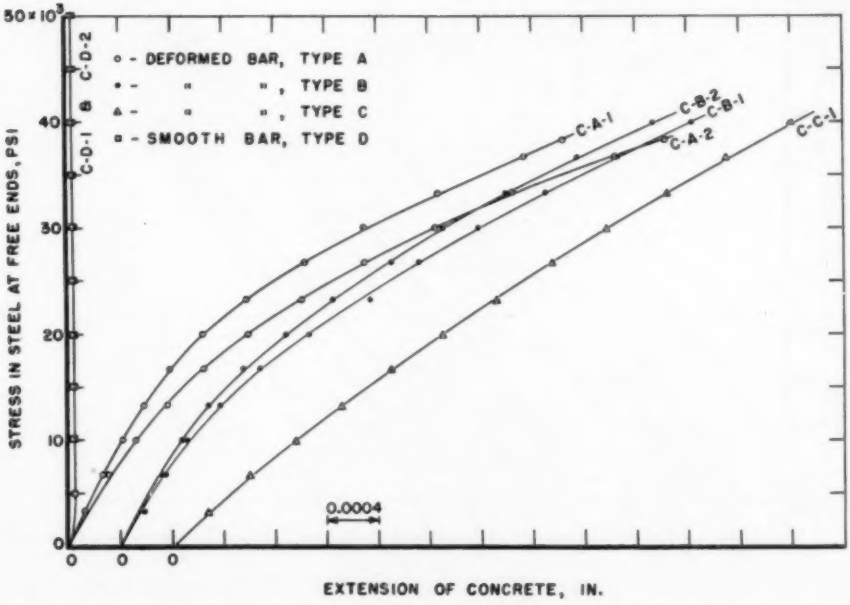


Fig. 4—Effect of surface characteristics of reinforcing bars on extension of concrete specimens ⅜ in. from the reinforcing bars

Distribution of strain on the surface of bond specimens

One pair of specimens containing ⅞-in. Type A bars was tested to determine the strain at the midpoint and the average strain on two opposite faces of each concrete specimen. The results of these measurements are given in Fig. 7. While variation of strain at the midpoint of the specimen is somewhat erratic, all four gages placed at these points indicated slight tensile strains (10 to 25 $\times$ 10^{-6}) up to an applied stress of 60,000 psi. Above this value of stress, the strain at the midpoint of the concrete became a compressive one reaching values of about 20 to 50 $\times$ 10^{-6} at an applied stress of 90,000 psi. The average strain on the sides of the specimens was somewhat more erratic. While in Specimen D-A-1 the average strain on both sides was a compressive strain up to an applied stress of about 80,000 psi, in Specimen D-A-2 one side indicated a tensile strain of about 60 $\times$ 10^{-6} and the opposite side indicated a compressive strain of about the same magnitude at an applied stress of 70,000 psi.

The presence of compressive strain at midpoint of the surfaces of the bond specimen confirms a similar phenomenon on the tensile faces of beams at points midway between successive cracks mentioned by Clark.[1]

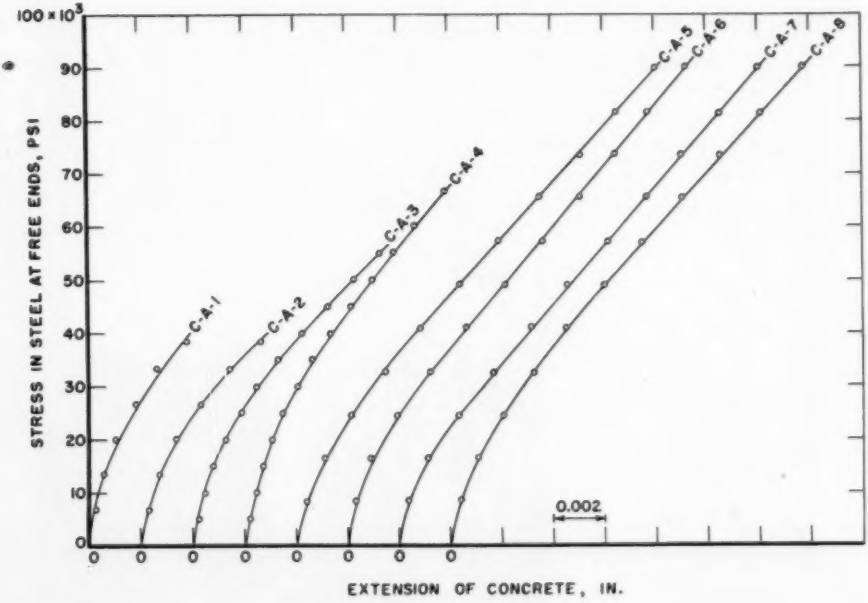


Fig. 5—Relationship between stress at the ends of the embedded bar and extension of concrete in tensile bond specimens for steels of different yield strengths

GENERAL DISCUSSION

Extensions of concrete and of the embedded bars observed in the bond specimens can be used to estimate the width of crack at the surface of the steel bars. The tensile bond specimen can be regarded as a segment of a long axially reinforced prism subjected to tension, with a crack spacing of 8 in. The crack width at the surface of the prism would then be given by the extension of the portion of the bar between two successive cracks assuming that the over-all length of the surface of each concrete segment remains constant. This assumption is substantially true as shown by the average strains on concrete faces of bond specimens given in Fig. 7.

To estimate the width of crack at the surface of the steel, the extension of concrete immediately adjacent to the bar must be estimated from the extension observed at a distance of $\frac{3}{8}$ in. from the surface of the steel. It will be assumed for this purpose that the extension of the concrete increased linearly from zero at the exterior surface of the concrete to a maximum at the steel's surface. Thus, the extension observed at a distance of $\frac{3}{8}$ in. from

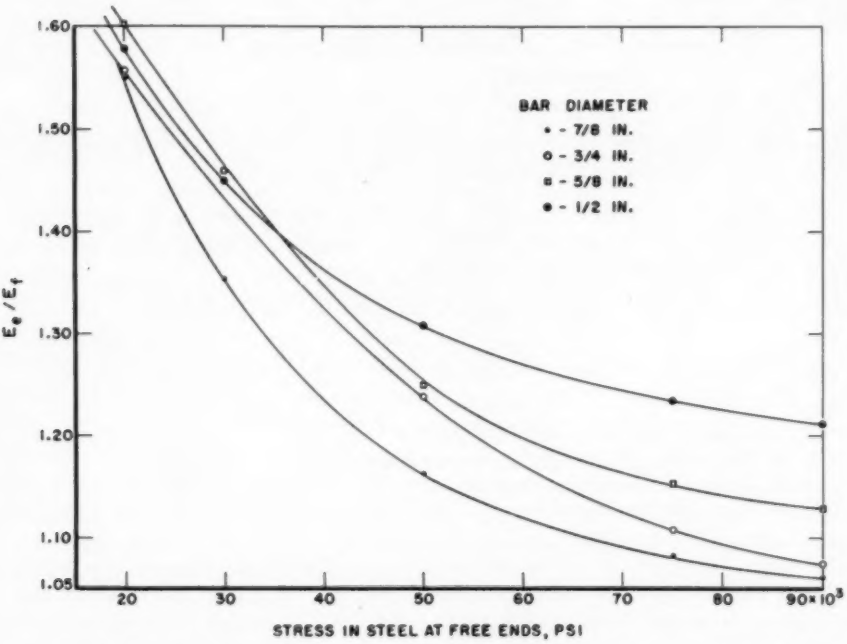


Fig. 6—Relationship between effective modulus of elasticity of embedded bar and stress at its ends, for deformed bars of Type A and for four different diameters with yield strengths from 99,000 to 104,000 psi. E_e = effective modulus of embedded bar; E_f = modulus of free bar

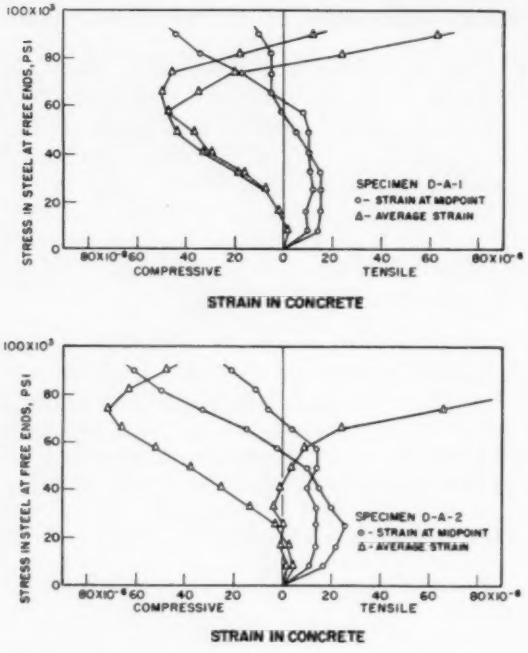


Fig. 7—Distribution of strain on exterior surface of bond specimens. Measurements were made on two opposite faces of each specimen

the surface of the steel must be multiplied by 1.17 to be converted into values of extension which might be expected at the steel surface.

The ratio of the estimated width of crack at the surface of the bar to the width of crack at the exterior surface of concrete is given in Fig. 8. This ratio was computed from the expression

$$\frac{W_s}{W_c} = 1 - \frac{e_c}{e_s}$$

where

W_s = width of crack at the surface of bar
W_c = width of crack at the exterior surface of concrete
e_c = extension of concrete adjacent to bar
e_s = extension of embedded bar

The single curve given in Fig. 8 is based on the average value of extensions of concrete observed in four specimens C-A-5, C-A-6, C-A-7, and C-A-8, and the average of two specimens S-A7-1 and S-A7-2 in which the extensions of steel were observed. All of these specimens contained a $\frac{7}{8}$-in. deformed bar of Type A with a yield strength of 104,000 psi, and the results shown in Fig. 8 may not necessarily be applicable to bars of other patterns and diameters and for other ratios of reinforcement.

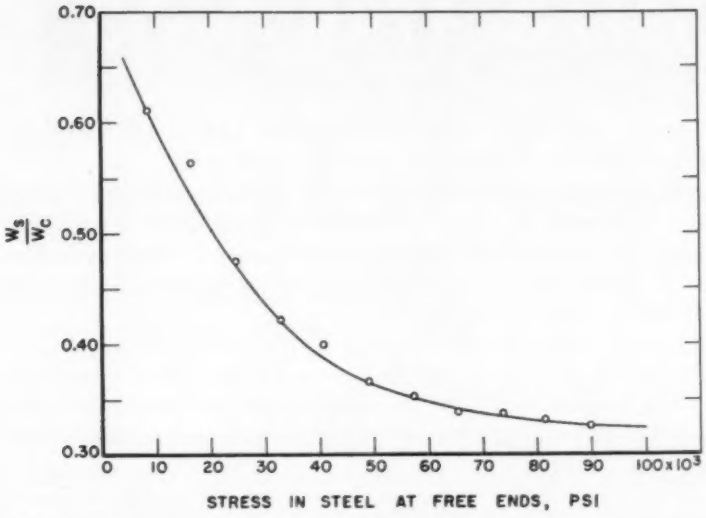


Fig. 8—Relationship between the ratio W_s/W_c and stress at the free ends of embedded bar. W_s = width of crack at surface of bar; W_c = width of crack at exterior surface of concrete. Bars of Type A, ⅞ in. diameter, yield strength 104,000 psi, and ratio of reinforcement 0.017

The curve in Fig. 8 indicates that for a steel stress of 20,000 psi the crack width at the surface of steel is 0.50 of that at the surface of concrete; at a stress of 40,000 psi this ratio is 0.39, and at a stress of 90,000 psi it is 0.33. This phenomenon explains the results reported by Kennedy[3] in his study of exposure tests of stressed reinforced concrete beams. Kennedy reported that in every case where specimens were alike, beams with bars having deformations complying with ASTM standard A 305 for deformed bars were superior in durability ratings to those reinforced with bars having old style deformations.

Data on the effective modulus of elasticity presented in Fig. 6 suggest that the tensile bond specimen affords the means of determining with precision the extent to which concrete participates in resisting tension in a reinforced concrete beam after formation of tensile cracks. The relationship between the effective modulus of steel and stress in the steel may be used to establish the variation in flexural rigidity of a cracked reinforced concrete beam and hence to formulate an expression for beam deflection in terms of the effective modulus of embedded steel reinforcement.

CONCLUSIONS

On the basis of tests of tensile bond specimens and measurements as described, the following results were observed, and conclusions drawn:

1. The extension of concrete adjacent to a well-designed deformed bar

was a significant fraction of the total extension of the steel bar. The extension of concrete for a smooth round bar was negligible. Thus, the tensile bond test serves to evaluate the effectiveness of bars in controlling cracks.

2. The relationship between extension of concrete and stress in steel was curvilinear up to a stress of about 40,000 psi. Above this stress, extension increased linearly up to the yield strength of the steel.

3. The average strain in the embedded bar was markedly less than that of a similarly loaded free bar. The "effective" modulus of elasticity of the embedded bar was 55 to 60 percent greater than the modulus of a free bar at a stress of 20,000 psi. The "effective" modulus approached the value for a free bar, as the steel stress increased.

4. Concrete strain at the midpoint of the tensile bond specimen was a slight tensile strain at first and then became a compressive strain at higher loads. The average strain on the sides of the specimen was quite erratic, at times indicating length changes of opposite signs on the opposite sides of a given specimen.

5. Crack width at the surface of the bar was assumed to be given by the difference between the extension of the embedded bar and that of concrete adjacent to the bar. It was estimated that the ratio of crack width at the bar surface to that at the exterior of surface concrete was 0.50 at an applied stress of 20,000 psi, 0.39 at 40,000 psi, and 0.33 at 90,000 psi.

6. Bars of Types A and B were of approximately equal effectiveness in controlling width of cracks, and the Type C bar was somewhat more effective in this respect.

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Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1959, for publication in the March 1960 JOURNAL.

CONCRETE BRIEFS

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Direct Formulas for Ultimate Strength Design of Columns with Symmetrical Reinforcement and Eccentricity in One Direction

By JAIME DE LAS CASAS*

The ultimate strength design method applied to columns with symmetrical reinforcement and eccentricity in one direction (most cases in usual practice) is easy and attractive. This brief discussion is intended as a summary of design procedure. It may also be of interest to those who want to adapt the ACI Building Code to foreign projects.

The notation used is similar to that in the appendix of ACI 318-56.[1] The following is added to simplify notation: in rectangular sections, $t_s = d - d'$; and when compression controls $f_s' = f_s - 0.85 f_c'$.

The following requirements are important in column design:

(a) Controlled concrete should be used [Section A602(f)]

(b) U (ultimate load) must be found according to Section A604.

$$U = 2 \text{ (basic load + live load)}$$

(c) All members subject to axial load shall be designed for at least a minimum eccentricity (see Section A608). Minimum eccentricities are:

Rectangular section	$0.1t$
Circular section	$0.05D$
Square section with round core	$0.05t$

DESIGN

Value of P_b

In a given section, the first step is to find P_b. If one accepts the criteria stated by Au[2] which are based on the ACI-ASCE Committee 327 report,[3] the P_b values should be:

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Rectangular section:

$$P_b = 0.36^* \left(\frac{90{,}000\dagger}{90{,}000\dagger + f_y} \right) (t + t_s) f_c' \, b$$

$$t + t_s = 2d$$

Circular section:

$$P_b = 0.36^* \left(\frac{90{,}000\dagger}{90{,}000\dagger + f_y} \right) (0.8 \, D + 0.75 \, D_s) f_c' \, D$$

Square section with round core:

$$P_b = 0.36^* \left(\frac{90{,}000\dagger}{90{,}000\dagger + f_y} \right) (t + 0.67 \, D_s) f_c' \, t$$

When $P_u > P_b$ compression controls

In this case the original formulas for P_u appear easier than any others. If the criteria of Whitney and Cohen[4] are accepted, f_y should be replaced by f_y'. Therefore the values for P_u will be:

Rectangular section:

$$P_u = \frac{A_{st} f_y'}{\dfrac{2e'}{t_s} + 1} + \frac{b \, t \, f_c'}{\dfrac{3te'}{d^2} + 1.18}$$

Circular section:

$$P_u = \frac{A_{st} f_y'}{\dfrac{3e'}{D_s} + 1} + \frac{A_g f_c'}{\dfrac{9.6 \, De'}{(0.8D + 0.67D_s)^2} + 1.18}$$

$$A_g = 0.785 \, D^2$$

Square section with round core:

$$P_u = \frac{A_{st} f_y'}{\dfrac{3e'}{D_s} + 1} + \frac{t^2 f_c'}{\dfrac{12 \, te'}{(t + 0.67D_s)^2} + 1.18}$$

When $P_b > P_u$ tension controls

Even though the original formulas for P_u are fully consistent, they appear cumbersome. By algebraic method it is easy to replace them by expressing A_{st} in terms of other quantities. For a rectangular section, this has been shown by Moyse.[5] It is possible to arrive at similar results for a circular section and square section with round core as follows:

Rectangular section:‡

$$A_{st} = \frac{P_u}{f_y \, t_s} \left[\frac{P_u}{0.85 f_c' \, b} + 2e' - t \right]$$

*The coefficient 0.36 must be reduced at the rate of 0.2 per 1000 psi (70 kg per sq cm) concrete strength in excess of 5000 psi (350 kg per sq cm).

†In metric system, replace 90,000 psi by 6300 kg per sq cm; all other units will be in kilograms and centimeters.

‡In rectangular section with symmetrical reinforcement: $A_s = A_s' = A_{st}/2$.

Circular section:

$$A_{st} = \frac{P_u}{f_y D_s}\left[\frac{P_u}{0.42 f_c' D} + 3.33e' - 1.48D\right]$$

Square section with round core:

$$A_{st} = \frac{P_u}{f_y D_s}\left[\frac{P_u}{0.57 f_c' t} + 3e' - 1.5t\right]$$

With the above formulas and keeping in mind the pertinent requirements of the appendix of ACI 318-56, it is possible to design columns with symmetrical reinforcement and eccentricity in one direction. It is important to call attention to the charts of Whitney and Cohen[4] and also those by Au[2] that offer a direct solution in columns with symmetrical reinforcement and are valuable design aids.

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Tensile Impact Tests for Concrete Reinforcing Steels

By RALPH G. CRUM*

It can be questioned whether the Charpy and Izod impact tests provide adequate information for direct stress impact design since they represent notch-band tests with complicated stress distributions. In fact, it can be questioned whether the Charpy and Izod tests give adequate information for any *rational* stress design, because the peak stresses set up during these impact tests are not known. On the other hand, the stress distribution is known, for the most part, in tensile impact tests. In this case, data can lead directly to rational direct stress design. In addition, the majority of individual metal fibers in all types of structures are under direct stress whether the loading is direct or not.

To evaluate the behavior of reinforced concrete beams under dynamic loadings, impact tests on concrete reinforcing steels are necessary. Concrete reinforcing steels are loaded essentially under direct stress and are correctly designed as such. It seems clear then that direct stress impact tests on concrete reinforcing steels would provide the most useful information.

Table 1 shows the results of tensile impact tests on three grades of reinforcing steel. Table 2 shows the chemical composition of these steels. The specimens were 0.252 in.

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TABLE 1—STATIC AND DYNAMIC PROPERTIES OF MATERIALS TESTED (AVERAGE VALUES)

Property	Type of loading	Hard grade steel	Intermediate grade steel	Structural grade steel
0.01 percent offset proof stress, psi	Static	70,000	40,000	40,000
	Dynamic	125,000	87,500	81,000
	Ratio of dynamic to static	1.78	2.18	2.02
Resilient energy, in.-lb per cu in.	Static	83	27	27
	Dynamic	264	128	109
	Ratio	3.18	4.74	4.04
Nominal ultimate strength, psi	Static	142,000	72,700	63,400
	Dynamic	221,000	128,500	93,500
	Ratio	1.56	1.77	1.47
Nominal fracture strain over 1 in. gage length	Static	0.155	0.37	0.44
	Dynamic	0.175	0.32	0.455
	Ratio	1.13	0.87	1.06
True fracture stress, psi, based on area and load at fracture	Static	181,000	132,000	125,000
	Dynamic	270,000	190,000	153,000
	Ratio	1.49	1.44	1.22
Reduction in area at fracture, percent	Static	29.0	58.0	65.5
	Dynamic	32.9	49.0	65.6
	Ratio	1.13	0.85	1.00
True fracture strain, $\ln \frac{A_o}{A}$	Static	0.33	0.84	1.06
	Dynamic	0.40	0.67	1.06
	Ratio	1.21	0.80	1.00

TABLE 2—CHEMICAL PROPERTIES OF MATERIALS TESTED

Reinforcement	C, percent	Mn, percent	Ph, percent	S, percent	Cu, percent	Si, percent
Structural grade steel	0.24	0.47	0.007	0.03	—	—
Intermediate grade steel	0.37	0.37	0.009	0.029	—	0.11
Hard grade steel	0.74	0.85	0.14	0.31	0.06	0.18

in diameter and the nominal gage length was 1 in. ($L/D = 4$). Impact loads were applied by a spring loader which maintained the velocity of impact throughout the tests at 450 in. per sec. The total length of specimens between grips was 2 in., thus giving a nominal strain rate of 225 per sec.* The total time of test from start of loading to fracture was about 1.5 milliseconds. The entire stress-strain curves were obtained during testing by the use of oscilloscopes and high speed motion picture photography.

As an example of useful information obtained in the tensile impact tests of reinforcing steels, it is shown that there was little change in ductility for the three grades of reinforcing steel under dynamic load as compared to static load. (The "static" strain rate was about 2×10^{-6} per sec.) In fact the hard grade steel showed an increase in ductility from 15.5 percent statically to 17.5 percent dynamically. While this increase is small, at least we do not have to be concerned about a *decreased* ductility for hard grade steel under dynamic loads. A fracture strain of 15 percent is probably adequate for reinforcing steel bars because the bond is broken and the concrete badly cracked before this strain is reached in the steel.† This, plus the fact that the already high ultimate strength for the hard grade steel bars of 142,000 psi is increased to 221,000 psi under dynamic load, seems to indicate that a further look into the possibilities of hard grade steel reinforcing for dynamic load is warranted.

*The strain rate in the reinforcing steel of a simply supported concrete beam 6 in. wide, 12 in. deep, on an 18 ft span would be in the order of 225 per sec if the beam sustained a uniform blast load of 1 billion lb per sec.

†Dynamic tests of concrete beams by Mavis and Greaves (ACI JOURNAL, V. 29, No. 3, Sept. 1957, pp. 233-252) indicate that the concrete disintegrates at 3 percent steel strain.

Increases in yield stresses and ultimate stresses under dynamic loads for the intermediate grade and structural grade reinforcing bars are also heartening. The greater ductility of these softer bars compared with the hard grade of reinforcing steel is a desirable trait in resisting dynamic loading. However, if it is not needed and is obtained at the expense of tensile strength (which contributes directly to the load capacity of the beam), then there may be justification for use of the harder grade of reinforcing steel. In any case the fact remains that further dynamic tensile tests would give good indication of the expected reaction of the reinforcing steel in concrete beams subjected to dynamic loads while the Charpy or Izod tests, with their stress raising notches, would not.

Cement Factor Related to Shape and Grading of Aggregate

By C. F. ZIETSMAN*

Proportioning by ACI's recommended practice for proportioning of concrete mixes, at a constant water-cement ratio, it was found that water requirements to produce concrete of a given slump could be predicted to within 1 gal. per cu yd of concrete by means of easy measurements on grading and voids of the aggregates.

This discussion is a sequel to a previous paper,[1] which was based on the older method of mix proportioning, and presents considerable improvements with respect to accuracy and simplicity of the equations. A study on the effect of shape of coarse aggregate on water demand of concrete is included in this work.

In Reference 1 the methods employed in this study were described in some detail. The only additional test employed is the measurement of voids in coarse aggregate per ASTM C 29-55T.[2]

In the previous study mixes were proportioned according to ACI (613-44),[3] because the work had commenced before the 1954 issue had come to hand. For the new study mixes were proportioned by the improved methods of ACI (613-54).[4] By using the empirical formulas of the previous study for guidance, the experimental work was greatly simplified.

In recognition of modern needs special attention was given to manufactured (crusher) sands, which comprised about half of the samples used.

To test the effect of shape of coarse aggregate, samples of ½-¾-in. stone were obtained from 28 sources.

NOTATION

G = standard deviation of particle size, or standard scatter of grains over successive mesh sizes. (In Reference 1 small Greek sigma was used)

V = proportion by volume of voids in dry-rodded coarse aggregate. V = 1 − solid content

v = voids in dry-shaken sand, Method 1 in Reference 1

W_4 = water required per cubic yard for 4-in. slump, lb

INFLUENCE OF G AND v WITH V CONSTANT

To test the influence of G and v on W_4, 55 samples of natural sand and 50 samples of crusher sand were used. Coarse aggregate consisted of crushed quartzite with a voids proportion of 0.4345. The sands varied from 1.75 to 3.6 in fineness modulus, from 0.6 to 1.8 in grading G and from 0.291 to 0.411 in voids v.

As a rule the water demand W_4 proved lower than expected by 10 lb per cu yd of

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concrete. This is attributed to a basic difference between ACI (613-44) and ACI (613-54). A careful study of these two methods shows that the latter allows for a smaller proportion of sand as the sand requires more water.

At the same time the curve family represented by Eq. (10) in Reference 1 showed a tendency to straighten out in the light of the new results. Since a straight line is more desirable than a curved line, an equation of the form

$$W_4 = Av + BG + C \quad \ldots\ldots\ldots (1)$$

was tried out and the best values of A, B, and C determined by methods of least squares. This resulted in an equation:

$$W_4 = 810.5v + 34.5(G-1) + 50.6 \quad \ldots (2)$$

In Fig. 1 the values of W_4 actually observed are plotted against the values derived from v and G using Eq. (2). The standard error of estimate comes to 6.8 lb of water per cu yd of concrete. This is much better than the previous values of 8.3 lb for natural sands

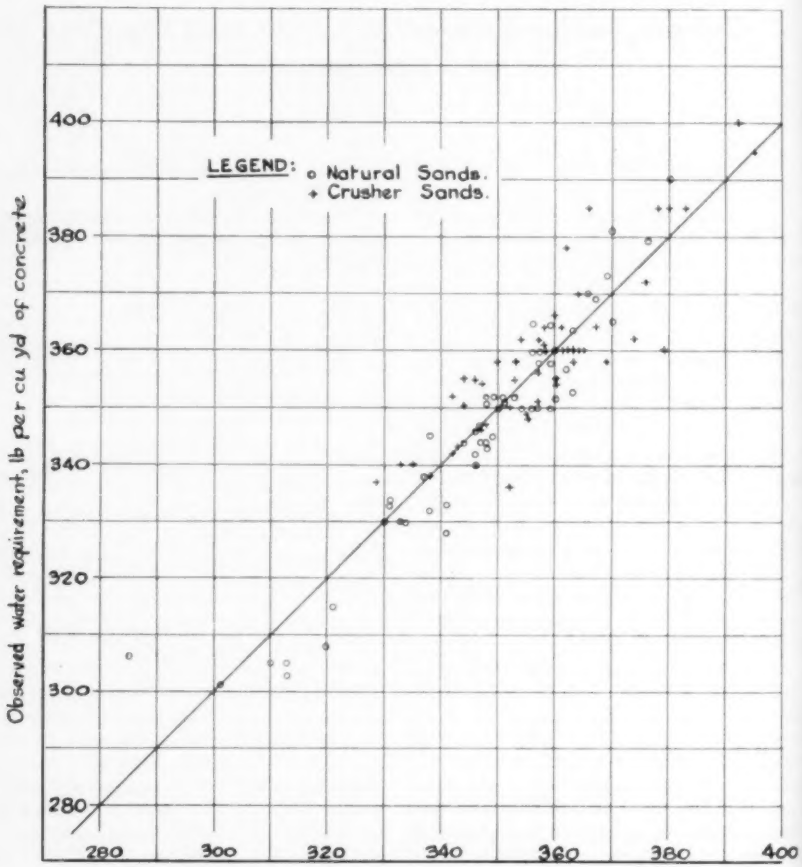

Fig. 1—Observed water requirement versus water requirement calculated by Eq. (2)

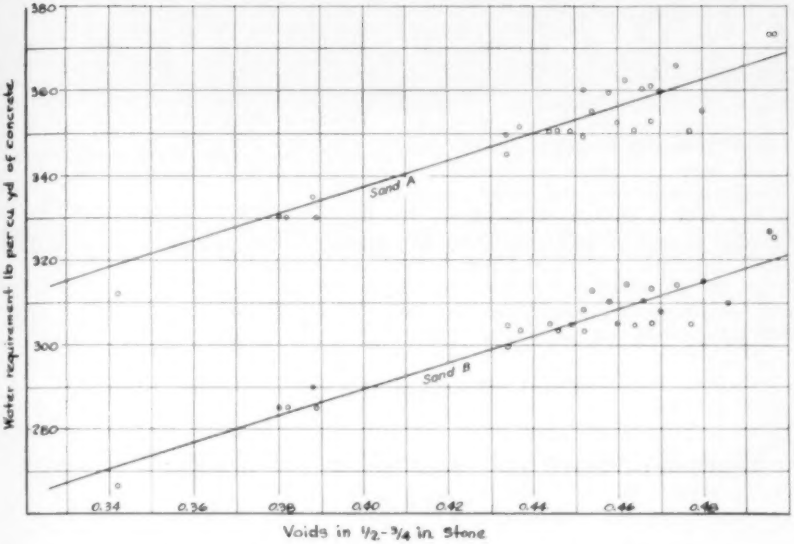

Fig. 2—Water requirement versus voids for ½-¾ in. stone

and 15 lb for crusher sands, and it is no longer necessary to treat crusher sand with a special reserve. While it is possible to obtain a better fit with a more complex equation, the sacrifice of simplicity cannot be justified.

So far particle shape has not been mentioned. Using equations from Reference 1, the shape factor can be derived from v and G. A study of the detailed results on the 105 samples shows that other things being equal, a normal variation in shape has more than twice the effect on water demand than does a normal variation in grading.

INFLUENCE OF V WITH v AND G CONSTANT

To test the influence of V on W_4, 28 samples of stone were used. The sand consisted of a well-graded crushed quartzite (Sand A in Fig. 2). As a check the tests were repeated using a well-graded sand of better particle shape (Sand B in Fig. 2).

For the purpose of this study all stone was screened between ½ and ¾-in. mesh, and V will have to be defined as the voids proportion in stone screened to uniform particle size.

Fig. 2 shows the water demand plotted against the voids in the stone V. The first point from the left represents hand-picked round stone. The next four points represent two samples of natural stone and two samples of specially processed crushed stone. The rest represent the bulk of crushed stone on the market.

Fig. 2 shows a constant increase in water demand with increase in stone voids irrespective of the type of sand used. W_4 increases by 31.56 as V increases by 0.1. Eq. (2) can therefore be modified quite simply to allow for shape of coarse aggregate as well. The comprehensive equation is:

$$W_4 = 810.5v + 34.5(G-1) + 315.6V - 86.5 \quad (3)$$

Eq. 3 is subject to a standard error of 8 lb of water per cu yd of concrete if the scatter observed in Fig. 1 and 2 is taken into account.

Using the corrections recommended in ACI (613-54) for size of stone, Eq. (3) has been found useful for graded as well as single-sized coarse aggregate, although the slump test loses its sensitivity for larger sizes of stone.

CEMENT FACTOR

Since the water-cement ratio is the dominant factor in the ultimate quality of concrete, it stands to reason that variations in water demand must be compensated for by directly proportional adjustment of the cement factor. In this study the water-cement ratio was maintained at 0.60, and an increase of 6 lb in water automatically necessitated an increase of 10 lb in cement. The cement factor varied by 2¾ bags per cu yd of concrete as from the best to worst aggregate combinations. Eq. (3) makes the cement factor predictable to within one-eighth of a bag.

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Technical Data Needed on Pneumatically Placed Mortar

By PAUL J. FLUSS*

During a recent survey the writer was appalled by the lack of test data pertaining to pneumatically placed mortar.

The only properties pretty well established are compressive strength and unit weight. Other data available are sporadic and somewhat contradictory. Recently Zynda furnished certain field data about water-cement ratio, absorption, air voids, and permeability in his paper "Shotcrete—Properties and Application," presented at the ACI 55th annual convention, Los Angeles. But the ever increasing use of pneumatically placed mortar as a structural material necessitates much more information such as:

1. Actual proportion at the gun and in place
2. Percentage and nature of air voids
3. Durability, as established by freezing and thawing tests
4. Shrinkage, etc.

Pneumatically placed mortar should come of age. It is time that the material and its use are placed on a technical basis. Just as it is commonplace knowledge what properties to expect when using concrete of a certain cement content the same information should be available for the different mix proportions of shotcrete.

There is a definite and urgent need for an extensive laboratory research program to be conducted by a large organization, which has the necessary facilities. And there is no doubt that the cement industry and contractors as well as governmental and private users will greatly benefit from such a project.

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Items appearing in this section can vary from 100 to 1000 words. It is an ideal place for those practical items generally considered too short to be submitted for publication, but containing material of potential importance to a great many.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Reinforced Concrete Design in the USSR

By C. W. YU,* MARGARET CORBIN,† and E. HOGNESTAD†

SOVIET CONCRETE DESIGN CONCEPTS

Design methods in the United States have entered a new era with the inclusion of ultimate strength design in the ACI Building Code of 1956. It is therefore of interest to American engineers to know the corresponding progress and development in the USSR. A brief outline of Soviet concepts of reinforced concrete design is given below as abstracted from Soviet documents. The fundamental philosophies on which the official Soviet design code of 1955 is based are discussed. Ultimate strength design is the only method used; the allowable stress theory has been abandoned. Translations of the references are available as indicated at the end of this article.

Limit conditions

Structures or structural members are designed in the USSR on the basis that service requirements will no longer be satisfied when certain limit conditions are reached. The concepts of design are therefore an advanced stage of ultimate strength design. Three limit conditions are generally recognized: (1) limit strength, (2) limit deformation, and (3) limit crack width. Accordingly, the purpose of design is to ensure, with a reasonable probability, that none of the three limit conditions will be reached during the entire service life of a structure. Different limit conditions may govern in different structures. For example, in a liquid-retaining structure, limit crack width may be more critical than limit strength. As far as limit strength (ultimate strength) is concerned, a single load factor, as commonly used in European and American practice, is considered unsatisfactory for the purpose of obtaining close agreement between design assumptions and actual service conditions.

Three different factors or coefficients are introduced in the Soviet design formulas.

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TABLE 1—DESIGN ULTIMATE STRESSES FOR VARIOUS SPECIFIED CONCRETE STRENGTHS[1]

Type and designation of stress condition	Concrete mix conditions	Design stresses of concrete in kg per sq cm for specified average cube strengths of								
		50	75	100	150	200	300	400	500	600
Axial compression (prism strength)	A	24	36	48	70	90	140	190	230	270
	B	22	33	44	65	80	130	170	210	250
Flexural compression	A	30	45	60	85	110	170	230	280	330
	B	27	41	55	80	100	160	210	260	310
Tension f_t	A	2.7	3.6	4.5	5.8	7.2	10.5	12.5	14.0	15.0
	B	2.4	3.2	4.0	5.2	6.4	9.5	11.0	12.5	13.5

NOTES:
1. From Reference 1.
2. Design stresses for concrete in tension, when alumina cement is used, are taken from Table 1 with a coefficient 0.70.
3. Design stresses shown in Line A are taken for plant-mixed concrete or concrete mixed on the job when equipment for automatic or semiautomatic batching of concrete components is provided (cement, aggregate proportions, water, and admixtures) with systematic control of concrete compressive strength and homogeneity. In all other cases values of concrete design stresses are taken from Line B.
4. When concrete quality is based on tension and systematic control of strength and homogeneity of concrete in tension is enforced, design stresses for concrete in tension f_t are taken from Table 1 and increased by 10 percent.

It is to be expected that the attainment of limit strength (ultimate strength) condition in a structure depends on many factors, the more important ones being:

(1) quality and mechanical properties of the materials used in the construction,
(2) nature of the external loads and other external actions,
(3) general service conditions of the structure, exposure, construction methods, etc.

Quality of materials

With respect to the influence of the mechanical properties of the structural materials on the limit strength of a structure, it is noted that the strength criteria depend on variability. Strength variation is observed from strength distribution curves which give the frequency of occurrence of any strength value. Therefore, the design strength expected of a material (average strength minus approximately three times the standard deviation) can be obtained

TABLE 2—LOAD FACTORS*

Designation of buildings and other structures	Specified load, kg per sq m	(lb per sq ft)	Load factors
Dead weight of the structure, with the exception of fill	—		1.10
Fill	—		1.20
Attic floors without special equipment (ventilation apparatus, water tanks, motors, etc.)	75	(16)	1.40
Apartments, hospitals (with the exception of vestibules and halls, in which crowds may be present), kindergartens, with conventional equipment	150	(32)	1.40
Dormitories, offices, classrooms, man-occupied industrial buildings with conventional equipment	200	(42)	1.40
Dining halls, restaurants, auditoriums, with conventional equipment	300	(63)	1.30
Hallways in dormitories, offices, and man-occupied space in industrial buildings	300	(63)	1.30
Halls and lobbies of theatres, cinemas, clubs, schools, stations, assemblies, sales rooms in stores	400	(85)	1.20
Floors of industrial buildings, warehouses, museums according to actual load, but not less than	400	(85)	1.20
Book storage, archives, viaducts according to actual load, but not less than	500	(105)	1.20
Stairways, vestibules, terraces, balconies	400	(85)	1.40
Hydrostatic pressure of liquids	—		1.10
Vertical and horizontal crane loads	—		1.30
Pressure of granular materials according to actual load	—		1.20

*From Reference 2.

by multiplying the specified strength (average strength) by a coefficient, called *uniformity factor*, and determined from strength distribution curves. Since the strength variability of a material is to a large extent dependent on the human control of the technological process of its manufacture, the uniformity factor is different for different materials, and depends on the control measure involved. As an example, design strengths for various specified concrete strengths are given in Table 1.

TABLE 3—SERVICE FACTORS*

Condition	Service factor
Flexural elements of precast structures	1.10
Compression members smaller than 12 in. in lateral dimension	0.80
Internal bays of slabs surrounded by integrally cast beams	1.25
Reinforcement bent into stirrups or bent-up bars	0.80
Tension and compression reinforcement made by coldworking steel	0.65
All normal cases not specifically covered by specification for design	1.00

*From Reference 1.

Load factors

In the design of a structure, the service live load is specified. However, this specified live load also has variability, as has the strength of a structural material. To obtain the maximum load that the structure may have to carry (ultimate strength design load) it is therefore necessary to multiply the service load by a coefficient referred to as *load factor*. As shown in Table 2, this load factor is different for different types of loadings considered. For example, if the live load under consideration is fluid pressure a low overload coefficient is used, and the coefficient increases, in general, with the degree of uncertainty involved.

Service conditions

The transition of a structure into limit condition depends also on the general service condition of the structure, taking into account favorable and unfavorable circumstances. A *service factor* is introduced in this respect. Table 3 indicates that this service factor may be greater or smaller than unity. Consider the case of a floor slab supported on edge beams. Although generally neglected in structural analysis, the torsional rigidity of the edge beams contributes considerably to the edge restraint of the slab, and consequently increases its ultimate strength capacity. This favorable effect of the beams on the slab calls for a service factor greater than unity. On the other hand, members of small dimensions present difficulty in steel and concrete placing. A service factor less than unity is used to take care of this unfavorable construction condition.

Design example

To illustrate the part these three safety coefficients play in design, the simple case of a beam in bending is considered. The limit strength of a beam section, in Soviet as well as American design procedures, is expressed as follows:

$$M_{ult} = p\,bd^2 f_y \left(1 - K\,p\frac{f_y}{f_c'}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where K is a numerical constant equal to about 0.60.

When the three safety coefficients are used, Eq. (1) becomes:

$$\Sigma (M\,n) \leqq m\,pbd^2\,k_s\,f_y \left(1 - K\,p\,\frac{k_s f_y}{k_c f_c'}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where
M = moment due to external live and dead load
n = overload factors for the external loads
m = service factor
k_s = uniformity factor for the reinforcement
k_c = uniformity factor for the concrete in flexural compression
f_y = specified average yield strength of the reinforcement
f_c' = specified average ultimate compressive strength of concrete
p = reinforcement percentage
b = width of section
d = effective depth

A section designed by Eq. (2) is expected by Soviet scientists and engineers to effect a close agreement between design assumptions and the actual service conditions of the structure.

Limit design

The 1955 Soviet building code recognizes the inelastic behavior of redundant structures and recommends design formulas for continuous structures, taking into account moment redistribution. For example, intermediate support moments of continuous beams are taken as $wl^2/16$. Sections designed in this manner are reinforced in such a manner that redistribution is assured without local failure.

CONCLUDING REMARKS

The Soviet design specifications and the officially prepared explanatory comments thereto are thorough and lengthy documents. It is evident that good progress is being made in this field in the USSR, just as it is in this country. Those who are familiar with advanced American literature on the subject will find a close parallel between our lines of thought and those of the Soviet engineers. The writers feel, in general, that a study of the translated documents was a thought-provoking experience, and should also be of interest to other engineers associated with research and development in the structural concrete field.

REFERENCES*

1. "Instructions for the Design of Sections of Reinforced Concrete Structural Elements, I 123-55/MSPMChP," Ministry of Industrial Construction, State Publishing Office of Literature for Structural Engineering and Architecture (Moscow), 1956, 172 translation pages.

This publication contains the full text of the 1955 Russian design code, "Standards and Specifications for the Design of Concrete and Reinforced Concrete Structures, NiTU/123-55,"

*English translations of the following references, made by the writers, have been placed in the translation pool of the Special Libraries Association. Photo-copies may be purchased from the John Crerar Library, 86 East Randolph Street, Chicago 1, Ill.

and also official explanatory comments, design tables, and examples of design problems. Approved by the State Committee on Construction of the Council of Ministers, USSR, for use by all ministries and organizations, this design code is based exclusively on ultimate strength design and limit design methods. The straight-line theory has been abandoned.

2. Goldenblatt, I. I., "Fundamentals of Structural Limit Design," State Scientific Society of Structural Engineers, State Publishing Office of Literature for Structural Engineering and Architecture (Moscow), 1955, 38 translation pages.

This pamphlet is intended for the use of engineers and technical personnel. It presents the fundamental principles of "Limit Design" as used in the USSR, establishes a basis for load factors and other factors involved in safety concepts, compares the new methods with older allowable stress methods, and reviews the basic computation principles of limit design.

3. Tal, K. E., "Limit Design of Plain and Reinforced Concrete Structures," State Scientific Society of Structural Engineering, State Publishing Office of Literature for Structural Engineering and Architecture (Moscow), 1955, 109 translation pages.

Intended for the use of engineers and builders, this pamphlet presents the principles of limit design for plain and reinforced concrete structures. Data required in design computations are given, such as uniformity factors, specified stresses in concrete and reinforcement at ultimate strength, design formulas for the various limit conditions, and design tables.

4. Polivanov, N. O., "Reinforced Concrete Bridges," Moscow, 1956. Translation of Chapter III "Standards of Strength Design of Reinforced Concrete Bridges," 7 pp.

This chapter outlines the principles involved in ultimate strength design of bridges in the USSR. Particular emphasis is given to overload factors and uniformity factors.

Bridges

Concrete bridges (in Croatian-Serbian)

M. Trojanovic, *Naše Gradevinarstvo* (Belgrade), V. 12: No. 3, Mar. 1958, pp. 49-54; No. 4, Apr. 1958, pp. 79-84

Reviewed by J. J. Polivka

Part 1 discusses fundamental problems of massive bridges, their basic history, progress, and developments. History of bridges goes back 5000 years and the seven important periods described specifically are: pre-Roman, Roman, Middle Age, Renaissance, 18th and 19th century, 1900-1930, and 1930 to the present. Refers to bridges in brick, stone, plain concrete, and reinforced and prestressed concrete. Ideas and opinions of leading bridge engineers and architects are cited.

Part II deals with the principles of conception, design, and construction of bridges from the following standpoints: (1) objectivity, (2) functionalism, (3) stability and safety, (4) rationality and solidity, (5) inventiveness and originality, and (6) aesthetics. Author refers to the opinion and criticism of recognized experts, most of them already mentioned in Part 1. He concludes with recommendations for further progress in design and construction of bridges, namely: (1) education of young engineers should also include knowledge in the fields of general culture, arts, and aesthetics; (2) thorough study of regulations and building codes; (3) better understanding of basic principles by all individuals involved and; (4) necessity and importance of the unit should never be overlooked. The builders with proper education, knowledge, and experience should have the freedom to apply their own ideas

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in the construction, subject to the approval and supervision of the designers.

Structural behavior of a plate resembling a constant thickness bridge abutment wingwall

H. P. Harrenstien and W. C. Alsmeyer, *Bulletin* No. 182, Iowa Engineering Experiment Station, Jan. 1959, 24 pp., $0.75

Reports on the first two phases of a research project on concrete bridge abutment wingwalls of the flared type. Phase 1 includes a literature review and questionnaire survey. Phase 2 includes the results of a study to establish a feasible analytical method for the solution of this type of plate problem.

Moment contours for bridge abutment wingwalls of constant thickness

C. L. Hulsbos and W. H. McCasland, *Bulletin* No. 183, Iowa Engineering Experiment Station, Feb. 1959, 28 pp., $0.75

Bulletin No. 182 reported the first two phases of this project (see review above). This bulletin presents design data for wingwalls of constant thickness. These design data are in the form of moment contours for wingwalls of the various proportions being used by the Iowa State Highway Commission.

Early American bridges

Richard Sanders Allen, *Consulting Engineer*, V. 11, No. 4, Oct. 1958, pp. 112-116, 118, 120, 122
Reviewed by Aron L. Mirsky

Nostalgic illustrated article, recommended reading for the structural engineer interested in the history of his profession and for any other historically-minded person as well. [cf Gravel, L. P., "Historique des pontsroutes daus la province de Quebec," *L'Ingenieur* (Montreal), V. 43, No. 170, 171, Summer, 1957, pp. 11-17, Autumn, 1957, pp. 31-36; "Current Reviews," ACI Journal, V. 30, No. 7, Jan. 1959, (*Proceedings* V. 55), p. 817.]

Reconstruction of the Chazey Bridge on the Ain River (Reconstruction du pont de Chazey sur la rivière d'Ain)

J. Courbon, Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 129, Sept. 1958, pp. 977-998
Reviewed by Henri Perrin

The Chazey Bridge is a continuous prestressed concrete bridge. The side spans are 136 ft long and the center span 187 ft. The prestressing system, developed by the contractor (system GTM), is also described: its outstanding feature is the anchorage which prevents any slippage.

The erection by successive cantilevering is extensively described and illustrated.

Summary of replies to questionnaire on preservation and repair of concrete bridges

HRB Committee on Bridge and Culvert Maintenance. *Circular* No. 382, Highway Research Correlation Service, Highway Research Board, Mar. 1959, 22 pp.

A survey was conducted in 1956 in which pertinent questions relating to the preservation and repair of concrete bridges were posed to state highway departments. This circular summarizes the replies. The questions covered types of deterioration, preventive maintenance, materials, and repair methods.

Reconstruction of the Lisle-sur-Tarn bridge (La reconstruction du pont de Lisle-sur-Tarn)

J. Courbon, *Annales des Ponts et Chaussées* (Paris), V. 128, No. 5, Sept.-Oct. 1958, pp. 561-581
Reviewed by Aron L. Mirsky

Structure, replacing an old suspension bridge, is of interesting form: a two-hinged concrete arch of 94 m span, with the central portion (56.4 m) of the deck integral with the arch and the remainders (22.5 m each) continuous with this central portion and extending (without intermediate columns) to simple supports at the abutments, thus spanning 101.52 m between end bearings. In cross section, bridge consists of two thin-walled rectangular tubes ("caissons") with the deck forming the roof slab in the central portion and cantilevering out on either side, for a total deck width of 9.58 m. Economy of this type of section is particularly noted. Centering was also unusual and author states this represented the only really difficult portion of the project: a two-hinged timber arch consisting of ten trusses fabricated with nails, strongly braced.

Conception, design, construction, and method of analysis of this triply-indeterminate structure are covered briefly.

Construction

Intricate footings, ramps complicate garage project

Contractors and Engineers, V. 56, No. 4, Apr. 1959, pp. 146-149

Discusses the construction of a seven-story, reinforced concrete, self-parking garage. Unusual features are 190 in. bell-bottomed footings and warped floor slabs. The footing has an 80 in. shaft and is capable of supporting 1,760,000 lb. The 15 in. floor slabs have dished out curves for the car ramps and posed special forming problems which are described.

Reconstruction of Queen's Quay, Belfast

R. J. N. Sweetnam *The Structural Engineer* (London), V. 36, No. 4, Apr. 1958, pp. 129-135
Reviewed by C. P. Siess

Includes description of precast reinforced concrete piles and beams in dock reconstruction.

Technical data on Soviet construction machinery from the Bulletin of Technical and Economic Information

JPRS (NY) *Reprint* No. 790, Nov. 10, 1958, 26 pp. Translation of *Byulleten' Tekhniko-Ekonomicheskoy Informatsii* (USSR), 1957, No. 2, pp. 68-69, 70-72; No. 3, pp. 75-76; No. 4, pp. 71-72, 74-76; No. 5, pp. 73-75; No. 6, pp. 61-62; No. 8, pp. 69-70; No. 9, pp. 69-71, 76-77; No. 11, pp. 69-70, 75-77; No. 12, pp. 57-58, 63-64. (*Translation* 59-11026 available from Office of Technical Services, Department of Commerce, Washington 25, D. C., $0.75)
Technical Translations V. 1, No. 3, Feb. 6, 1959

Contains data on construction-materials machinery, road machinery, and dredges. Data are included on a mobile mixer with self-propelled loader, crane loaders for use at reinforced concrete products plants, and the production of concrete block on mobile machines.

Closure of Haringvliet sea arm (in Dutch)

A Spoel, *Cement* (Amsterdam), No. 23-24, 1958, pp. 920-927
Reviewed by John W. T. Van Erp

This is part of the enormous Delta plan, which aims at the closing of several sea arms, thereby strengthening the system of flood defense in the Netherlands and adding to its land area. Many large concrete structures are part of the project. The above item is a 2 mile long dike incorporating a 0.6 mile long concrete structure consisting of a group of discharge sluices (steel segmental gates). A rectangular construction pit was built in tidal water using large concrete caissons floated in for final closure.

Concrete construction as a base for flooring

Joseph J. Shideler, *Installation and Maintenance of Resilient Smooth-Surface Flooring*, Building Research Institute, 1958, pp. 23-33

Attention is given to methods of producing concrete floors that can be covered readily with smooth-surface resilient floor coverings. Mix proportions, placing, finishing, and curing are discussed. The problem of moisture is discussed as well as the various types of concrete floors.

Most of the difficulties encountered with concrete as a base for resilient tile can be entirely eliminated by applying the fundamental rules for making good concrete. These include the selection of sound well-graded aggregate, the use of a well-proportioned mix with adequate cement and a minimum of water, good compaction, careful finishing at the proper time, and adequate curing.

St. Pius X Basilica at Lourdes (in French)

J. Chaudesaigues, *Travaux* (Paris), V. 42, No. 290, Dec. 1958, pp. 1003-1019; V. 43, No. 291, Jan. 1959, pp. 9-22

The new church is subterranean and oval in plan, about 200 m long and 80 m wide. Being underground and the layout of the church itself caused some rather unique and interesting problems that had to be solved.

By using prestressed concrete it was possible to construct the roof as a series of fairly flat portal frames of low depth at midspan whose supports are regularly arranged on a curve concentric with the outline of the Basilica.

Sport auditorium in Mulhouse (La Salle des Sports de Mulhouse)

T. Jean-Blocu, Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 129, Sept. 1958, pp. 999-1020
Reviewed by Henri Perrin

The roof of this auditorium is made of four barrel vaults intersecting at 90 deg, and sup-

ported on four points. The shells have an average thickness of 3¾ in. The ties balancing the horizontal thrusts are prestressed. Design and erection procedure are presented.

Buildings of the permanent headquarters of UNESCO in Paris (Les batiments du siège permanent de l'UNESCO a Paris)

B. H. Zehrfuss, L. Hahn, and R. Bernard, Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 11, No. 132, Series 27, Dec. 1958, pp. 1365-1402

Authors' Summary

The UNESCO headquarters consists of a y-shaped administration building with 700 to 800 offices and a conference building with a trapezoidal roof.

The article describes the method of construction adopted for the roof of the conference hall and for two canopies, one facing the avenue de Suffren and the other facing the Place Fontenoy. The first consists of a central arch supporting two unequal cantilevered shells; the second, more classical, consists of a slab stiffened by curved beams and supported by tapering columns.

For the construction, excavated material was first stockpiled to be used as fill and as a source of aggregate for concrete. No rendering was applied to the concrete which has been cast in blocks of uniform weight, with visible joints.

Dunlop factory at Amiens (L'usine Dunlop a Amiens)

M. L. Hahn, Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 11, No. 129, Series 48, Sept. 1958, pp. 1021-1034

Author's Summary

The workshop of the Société Dunlop is a 918 ft long and 131 ft wide structure covered with a reinforced concrete north-light shell roof. A box girder 918 ft long running the length of the building supports 102 north-light shells each of which is 18 ft wide and has a span of 65½ ft. The outer edges of these shells are supported by columns. Each shell consists of a 2¾ in. thick cylindrical concrete shell with a parabolic directrix, and is stiffened at its base by a gutter-beam and at the top by a rib at the upper edge of the corrugated reinforced glass partition. The design was carried out by calculating the shell as a beam subject to oblique bending. Construction included the use of ten reusable centerings.

Exhibition palace of the Centre National des Industries at Techniques (Le Palais des Expositions du Centre National des Industries et Techniques)

N. Esquillan, *Mémoires*, Société des Ingénieurs Civils de France (Paris), V. 112, No. 6, Nov.-Dec. 1958, pp. 484-500

Exhibition palace of the Centre National des Industries et Techniques (Le Palais des Expositions du Centre National des Industries et Techniques)

P. Calfas, *Le Génie Civil* (Paris), V. 135, No. 15-16, Aug. 1-15, 1958, pp. 325-333

Centre National des Industries et des Techniques

The Engineer (London), V. 206; No. 5365, Nov. 21, 1958, pp. 821-824; No. 5366, Nov. 28, 1958, pp. 862-864

Reviewed by Aron L. Mirsky

Descriptions of a most interesting exhibition center, which, besides holding two world's records, is serving as the focal point for the revitalization of the area in which it is situated.

Roof of structure, based on a design by Esquillan, is a thin corrugated double shell formed by three interpenetrating catenary cylinders, inscribed in an equilateral triangle 218 m on a side, and supported at the three points only. It is thus the world's longest span shell structure (206 m span at each façade, 238 m at the groins), and the structure having the world's largest surface supported at one point (7000 sq m plan area, 9000 sq m roof area). Each façade is fully glazed, and a 5-story rectangular building is built into each façade; the floors in

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the central portion of the structure are of triangular design. Exhibit area is 71,000 sq m, plus 30,000 sq m of service areas. Public access for 60,000 persons at one time is by 124 doors (there are 7 entrances for freight, exhibits, etc.). While the shells were cast in place, extensive mechanization and prefabrication resulted in substantial completion of the facility in the short time of 21 months.

Esquillan's paper contains much interesting material on the conceptions (both design and analysis) and the preliminary designs as well as on the final design details.

(See also Balency-Bearn, M. A., and Lacombe, M. G., Supplement to *Annales*, Institut Technique du Batiment et des Travaux Publics (Paris), No. 127-128, July-Aug. 1958, pp. 876-902; "Current Reviews," ACI JOURNAL, V. 30, No. 7, Jan. 1959 (*Proceedings* V. 55), pp. 818-819.

Construction Techniques

Modern forming methods — form panels, ties, and scaffolding in theory and practice (Moderne Verschalungsmethoden. Schalplatten, Verbindungsmittel und Montage in Theorie und Praxis)

Die Bauzeitung (Stuttgart), V. 63, No. 9, Sept. 1958, pp. 422, 424, 426, 428

Reviewed by ARON L. MIRSKY

Describes some of the newer methods of form construction, with three examples consisting of recent German projects (a school, a hospital, and a municipal swimming pool).

Lift slab economics

DONALD J. WAKELY and ORUS O. EASH, *Consulting Engineer*, V. 11, No. 6, Dec. 1958, pp. 82-83

Reviewed by ARON L. MIRSKY

Comparison of unit costs for three types of floors (concrete on structural steel, cast-in-place concrete, and lift slab) for a typical 10-story building with 100,000 sq ft of floor area indicates little difference between the first two, considerable savings with the last. Additional savings possible with lift slab construction, and lift slab limitations, are also noted.*

*cf. RICE, EDWARD K., "Economic Factors in Prestressed Lift-Slab Construction," ACI JOURNAL, V. 30, No. 3, Sept. 1958 (*Proceedings* V. 55) pp. 347-357.

External vibration of concrete

J. KOLEK, *Civil Engineering and Public Works Review* (London), V. 54, No. 633, Mar. 1959, pp. 321-325

AUTHOR'S SUMMARY

The influence of acceleration, amplitude, and frequency is discussed and a generalized theory of compaction is suggested. An experiment was conducted to find the influence of the frequency of vibration on surface finish. Practical recommendations are quoted for external vibration including guiding points for the estimation of the type and the number of vibrators needed.

Three steps to correct surface defects

Concrete Construction, V. 4, No. 5, May 1959, pp. 6-8

When a freshly finished concrete surface shows defects such as fins, form tie holes, and honeycombing, certain steps can be taken to conceal these imperfections. This article reviews those steps and includes a series of excellent photos which cover the several techniques involved.

Experiences with current wall constructions (Erfahrungen mit neuzeitlichen Wandbauarbeiten)

WERNER PFEFFERKORN, *Die Bauzeitung* (Stuttgart), V. 63, No. 7, July 1958, pp. 333-336

Reviewed by ARON L. MIRSKY

Basic discussion of strength, weight, cost, crack-control, moisture-proofing, and other aspects of wall construction, primarily for dwellings. Use of concrete block and concrete-covered walls is emphasized.

Factory building for AKU Research Institute in Arnhem (in Dutch)

F. J. B. BARENDS, *Cement* (Amsterdam), No. 23-24, 1958, pp. 959-965

Reviewed by JOHN W. T. VAN ERP

Factory consists of 6 story office building, 5 story chemical building and test-hall, all concrete construction is on pile foundations, much of it prefabricated. Test-hall has prestressed portals of 147 ft span, 32 ft clear height. Portal members incorporate light system with prestressing steel in walls. Office wing has prestressed floor beams of 42 ft span. Much of construction was during freezing weather, concrete temperatures were maintained by steamheat under canvas cover.

A primer on vibration

Concrete Construction, V. 4: No. 3, Mar. 1959, pp. 1-4; No. 4, Apr. 1959, pp. 1-3

Part 1 deals with the fundamentals of consolidating concrete. Three of the four general types of concrete vibrators are described briefly: internal, form, and surface; table vibrators are not covered. Hints are given on how to choose a vibrator; factors to consider are type of power, frequency, amplitude, and force. Selecting the right slump for any given job will depend on several factors, including vibratory equipment efficiency, depth and width of the section, amount of reinforcement, and available handling equipment.

The second installment covers job site practices which will assure the best results. Both correct and incorrect practices are covered.

New way to cool aggregates—by steam

Engineering News-Record, V. 162, No. 14, Apr. 1959, pp. 38-42

The Hartwell Dam on the Savannah River, Ga., is being constructed with the use of cooled aggregate. The cooling plant has five aggregate tanks and reduces the temperature of the aggregate to approximately 40 F in about 45 min. This is done by enlisting a venturi tube through which steam is jetted thus lowering the pressure; this causes a partial vacuum in the tank which induces the moisture on the rocks to vaporize. This vaporization absorbs heat from the aggregates and thereby cools them. The speed of cooling is fast enough to enable a concrete-placement rate of 250 cu yd per hr.

Paper tapes for concrete

Concrete Construction, V. 4, No. 1, Jan. 1959, pp. 10-12

Describes ways in which paper and masking tape have been used in concrete construction. Tapes can be used to seal the edges of paper cartons employed to create structural voids; tapes can be used to mask finished concrete surfaces, thus protecting them from splashing when placing fresh concrete; tapes can be used to construct two colored concrete floors since the masking application prevents one color from bleeding into the other.

Manual of recommended practice for cast-in-place, exposed, architectural concrete finishes

Concrete Industry Board, New York, 1958, 15 pp., $1

The manual is prepared in specification form to be used either in part or in its entirety to simplify obtaining "smooth surfaces, lack of blemishes, elimination of fins, honeycombs, etc." Each specification section is accompanied by explanatory comments. The sections cover general recommended practices, materials, formwork, reinforcement, placing concrete, stripping and curing, finishing, and samples.

External rendered finishes

Concrete Construction, V. 3: No. 10, Oct. 1958, pp. 4-8; No. 11, Nov. 1958, pp. 13-15; No. 12, Dec. 1958, pp. 8-10, 12

External finishes for the walls of concrete buildings may be either structural or painted finishes. This series dealt with the rendered types of finish applied either by hand or mechanical means.

Part 1 covered types of treatment, appearance of finishes, maintenance requirements, and conditions of exposure. Part 2 dealt with the materials used in external rendering and the types of background materials on which the rendering is applied. The final installment covered mix proportions and the specific techniques called for to produce various types of finishes.

Natural concrete

Concrete Construction, V. 4, No. 4, Apr. 1959, pp. 6-7

A new technique for obtaining an original finish on site-cast concrete has been developed by a Norwegian architect. It might be described as a refinement of the intrusion method of placing concrete, coupled with sandblasting. The technique has been used on a number of small buildings and on a 17-story office building.

New forming method, cap-form support cuts work time on bridge piers

Bill Allen, *Contractors and Engineers*, V. 65, No. 5, May 1959, pp. 84-87

A method for forming box-shaped stems for piers on a double bridge in Oklahoma City has resulted in a reduction of labor costs to one-third below what was anticipated.

This forming method consists of placing a side panel and two end panels on a jig. The side panel lies flat allowing for more convenience in placing the reinforcing steel. When the steel is set the top panel is lowered into position to close the box. The box is then swung into an upright position by the jig and a crane lifts it onto the pier and its ready to receive concrete. An average box form size is 10 x 4 x 16 ft.

Another cost saver was the diagonal supporting timbers that braced the box forms. These timbers were supported by the pier stem itself and eliminated eight false work piles.

Little gimmicks mean big savings on Biscayne Bay crossing

Engineering News-Record, V. 162, No. 13, Apr. 1959, pp. 44-48

A 3.2 mile long causeway, which includes two bridges totaling 3288 ft is being constructed between Miami and Miami Beach.

New construction methods being employed are: (1) floating steel walers to support pile cap forms; (2) pile driving templates, 22 ft wide by 90 ft long, that can accommodate 42 piles at one time; (3) plastic "sausage casing" to keep grout from being washed away; (4) traveling gantry, designed to move from side to side on a track, used to place 42-ton beams and; (5) a floating concrete plant with a capacity of 25 cu yd per hr.

Dams

Analysis of arch dams (in French)

V. Bauzil, *Travaux* (Paris), V. 43: No. 291, Jan. 1959, pp. 1-7; No. 292, Feb. 1959, pp. 84-88
Author's Summary

Starting with the work published by Haegelen (*Travaux*, Dec. 1954) and by Perkins (*Proceedings*, ASCE May 1952), the author completes the earlier results by calculating the statically indeterminate restraints which must be added, at the crown of the arch, to those obtained in the case of complete fixity at the supports to take account of the deformation of the foundation soil, and by calculating, for the same conditions, the complementary radial displacement to be added to that obtained for the section at the crown to pass from the case with complete fixity at the supports to the case with deformable supports:

(1) By calculating the radial displacement of an arbitrary point of the arch, when that of the section at the crown is known, for a uniformly distributed radial load and for complete end fixity or elastic support;

(2) By calculating the statically indeterminate restraints at the crown and the radial displacement of the section at the crown in the case of triangular loads symmetrical with respect to the apex.

New dam structures designed and erected in Yugoslavia (in Croatian-Serbian)

D. Milovanovic, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 6, July 1958, pp. 139-144
Reviewed by J. J. Polivka

Describes four concrete dams now under construction, their design, and an experimental model analysis. Among the dams discussed are (1) an arch dam on the Idbar River, (2) a multiple arch dam near Rastovaca-Posusja, (3) a triple-arch dam near Citluk-Mostar, and (4) the Modrac Dam, a multiple arch type.

Measuring the structural behavior of Hungry Horse Dam

Joe T. Richardson, *The Engineer* (London), V. 205, No. 5325, Feb. 14, 1958, pp. 240-243
Reviewed by Aron L. Mirsky

Describes the 681 instruments installed in the dam, their locations, etc., and mentions the high-speed computer methods used to reduce the large mass of data. The first reaction of those interested in structural behavior—not necessarily of dams—will be an instinctive turning to the brief bibliography, to determine where more solid food may be obtained.

Fundamentals of dam construction (Konstruktionsgrundsätze im Talsperrenbau)

Friedrich Tolke, *VDI Zeitschrift* (Düsseldorf), V. 100, No. 28, Oct. 1958, pp. 1347-1354
Reviewed by Aron L. Mirsky

Brief summary of the varoius types of dams and where to use them, well illustrated with examples taken from recent projects throughout the world.

Design

Progress, success, and future development of design with reinforced concrete in Switzerland (Fortschritt, erfolg und die zukuenftige gestaltung des stahlbetons in der Schweiz)

MIREO ROS, *Zement und Beton* (Vienna), No. 11, Feb. 1958, pp. 1-16

Reviewed by FRITZ KRAMRISCH

A discussion of developments in present design, stress relations, code requirements, and safety factors of reinforced concrete in Switzerland. The interesting remarks are accompaneid by many illustrative diagrams.

Theory of prismatic folded plate structures

J. E. GOLDBERG and H. L. LEVE, *Publication*, International Association for Bridges and Structural Engineering (Zurich), No. 17, 1957, pp. 59-86

APPLIED MECHANICS REVIEWS Jan. 1959 (A. L. ROSS)

The subject structure is defined as one composed of rectangular plate components connected along parallel edges and closed at the end by an end diaphragm or bulkhead. The purpose of the paper is to determine the stress distribution, and the moments and forces arising at the joints, due to enforcing continuity at these junctions. Because of the two-dimensional aspect of the structure as opposed to a one-dimensional truss structure, influence functions are generated instead of influence coefficients.

The influence functions are found by expanding the displacements and rotations of the edge of the rectangular plate components into a Fourier series and finding the forces at the junction resulting from unit Fourier components of the edge displacements and rotations. The effect of rotating the edge of the rectangular slabs, and deflections of these edges normal to their original plane are found from plate theory. The effect of deflections of the edge in the planes of the component plates is found from two-dimensional elasticity theory. These influence coefficient functions are tabulated.

A sample three-plate structure is analyzed using the first three symmetrical modes of the displacement Fourier series.

Reviewer notes that the method would become quite unwieldy if a larger number of Fourier terms would have to be included, although this difficulty could be eliminated by the application of digital computation techniques. Although the method is somewhat cumbersome, it is a solution of a complicated problem.

Application of the Laplace transformation to rheological computation of prestressed structures (in Polish)

C. EIMER, *Rozprawy Inzynierskie* (Warsaw), No. 6, 1958, pp. 1, 183-200

APPLIED MECHANICS REVIEWS Jan. 1959 (Z. BYCHAWSKI)

Assuming that the creep function depends on the difference between two arguments $C = C(t + \tau)$, which is equivalent to the assumption that creep is independent of the age of the concrete at the moment of applying the load, the Laplace transformation is applied to systems of integral equations derived for isostatic and hyperstatic prestressed structures. Thus the problem reduces to the solution of systems of linear algebraic equations, which facilitate considerably the attainment of effective results. The difficulties of inverse transformation are discussed. Possibilities of simplifications are indicated and illustrated by a numerical example of a two-hinge rectangular frame.

Tables for calculation of flat elliptical paraboloids over rectangular ground plan (Tafeln zur Berechnung des flachen elliptischen Paraboloids über rechteckigen grundriss)

BORIS KLIMOV, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 1, Jan. 1959, pp. 9-14

Reviewed by RUDOLPH SZILARD

Develops a numerical solution of the Laplace differential equations based on the method of finite differences. The flat shell surface covering a rectangular ground plan has been subdivided by means of a gridwork. Tables containing coefficients for the axial and shear forces facilitate the determination of the membrane stresses assuming free boundary conditions. The tables can be used for flat shells in the range of $f_a = 1/8a$ to $1/10a$, and $f_b = 1/8b$ to $1/10b$, where f_a and f_b represent the height of the end parabolas, $2a$ and $2b$ the dimensions of the rectangular ground plan, respectively.

Aerodynamic study of a cooling tower in the form of a hyperboloid of revolution (in French)

G. Golubovic, *Publication*, International Association for Bridges and Structural Engineering (Zurich), No. 17, 1957, pp. 87-94

Applied Mechanics Reviews Jan., 1959 (Author's Summary)

Author examines the effect of several parameters on the problem of the distribution of pressure around a hyperboloid of revolution subjected to wind loading. On the basis of the similitude theory, rules are worked out for the transfer to full scale of experimental results obtained on a model. The model constitutes a geometrical replica of a cooling tower of hyperbolic form. The parameters investigated are the geometrical shape and the location of the series of ribs placed on the outer surface of the tower.

Influence line ordinates at the 10th points of continuous beams—V. 2 (Zehnteilige Einflusslinien fuer durchlaufende Traeger. Band II.)

Georg Anger, Wilhelm Ernst and Son, Berlin, 1958, 276 pp., paper back—37 DM, hard back—41 DM

Reviewed by Fritz Kramrisch

The second volume of this well-known table work has been newly issued in a quality equal to that of V. 3, previously reviewed (Current Reviews, ACI Journal, Dec. 1956, p. 614).

This new enlarged volume contains the moments, shears, and reactions of beams continuous over two to five bays. It is divided into two parts: Part 1 considers the beams loaded with an over-all equally distributed dead load and a live load equally distributed for each bay; Part 2 considers any kind of loading.

Both parts are treated for three different end conditions; (a) both ends simply supported, (b) one end simply supported, the other fully restrained, and (c) both ends fully restrained. The span ratios between exterior and interior bays vary from 1:0.4 to 1:2.5. For equally distributed loads and up to three bays the values are given for span ratios varying by 0.01 which eliminates practically all interpolating. For beams having four and five bays and beams with general loading conditions the ratios vary by 0.1.

For all these cases of loading, end conditions, and span ratios, the tables give exact values of the maximum negative moments over the supports, maximum and minimum field moments, and shears and reactions. Each group of tables has an introduction with an example explaining the tables. Any experienced engineer, even without much knowledge of the German language, should be able to use most of the tables (equally distributed load and symmetrical span arrangement), because they are self explanatory. Only the more complicated tables require somewhat closer study of the explanations in the text.

Effect of symmetry and repetition in simplifying the analysis of structures

Simon Wooly, *The Structural Engineer* (London), V. 35, No. 9, Sept. 1957, pp. 357-363

Reviewed by C. P. Siess

Method is based on the resolving of asymmetrical load systems into symmetrical and antisymmetrical components, thus requiring analysis of only one half of a symmetrical structure. The author, however, continues this process of resolution of forces and splitting of the structure into smaller portions so long as each resulting portion is itself symmetrical or can be made symmetrical by the use of equivalent force systems. Illustrated with an example of a six-story, five-bay framework with lateral loads at all floor levels on one side.

Generalized slope-deflection and moment distribution methods for the analysis of viaducts and multi-bay, ridged portal frames

E. Lightfoot, *The Structural Engineer* (London), V. 36, No. 1, Jan. 1958, pp. 21-33

Author's Summary

Introduces two new methods of structural analysis for the single-story, multibay frame. The first, the generalized slope-deflection method, is developed from a consideration of the flexural properties of the curved (or bent) structural member. It is, however, restricted in its application here to those "linear" structures in which the connecting eave joints are only able to rotate and displace horizontally. The second, the generalized moment distribution method, is developed from the first method, and is restricted to the same type of structure. It is based on the physical conception of allowing the various eave joints, in turn, both

to rotate and displace horizontally together, so that the out-of-balance moments and thrusts are successively eliminated at these joints.

Theory and research in concrete shell design

A. M. Haas, *Proceedings*, ASCE, V. 85, ST4, Apr. 1959, pp. 1-21

Author's Summary

The design of shells by the classical theory is briefly reviewed to serve as a basis for presenting simplifications in design calculations. It is noted that shells are membranes having edge disturbances which lead to a simplified design procedure especially for shells of double curvature. To determine the effect of these disturbances, use has been made of the analogy between a shell curved in one direction and a beam on elastic foundations. This analogy is presented and provides a rapid means by which a design may be made.

Research data are presented and discussed. These data clarify the behavior of shells having various schemes of reinforcement, show the stiffening influence of edges, and illustrate types of failure. In the absence of a reliable theory of buckling, model studies of buckling phenomena should be made. The present trend is to use small scale inexpensive models. Attention is directed to the choice of model material in relation to concrete. The need for further research in shell design is emphasized.

A design method for composite tile-concrete floors

J. A. P. Laurie, *Bulletin* No. 13, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria, Mar. 1955, pp. 61-76

Author's Summary

A suitable method is presented for designing the reinforcement in composite floors of the hollow-tile type, in which joists built up from hollow burnt clay block are used to form the soffits for cast-in-place reinforced concrete ribs and topping. The proposed theories deal with bending stresses, shear and bond stresses, and ultimate strength in flexure; although primarily based on current reinforced concrete theory, they have, where possible, been checked against the results of experimental work performed at the National Building Research Institute.

Design of prismatic and cylindrical shell roofs

David Yitzhaki, Haifa Science Publishers, Haifa, 1958, 253 pp.

Reviewed by William C. Krell

This book emphasizes the need for a good comprehensive text on the subjects of cylindrical shells and folded plate design. It does not, however, meet this need. The terminology differs from the American concept and is somewhat vague and cumbersome to read.

The objectives of this book are reached by a device not unlike an application of virtual work; this device is termed "particular loading" by the author. Numerous specific cases are given and solutions presented. Some items, such as continuity, etc., are presented as fact, while actually we have no test data to confirm them as yet. No practical applications, reinforcing, or joint details, are given. This book could be a valuable addition to the library of someone acquainted with such design but is not recommended for the novice.

Joint displacements of framed plane structures (Cálculo de los desplazamientos de nudos de una estructura plana compuesta por barras)

Julian Navarro Gutierrez, *Bulletin*, No. 186, Instituto Técnico de la Construcción y del Cemento (Madrid), 1958, 44 pp.

Reviewed by Roger Diaz de Cossio

Using a vectorial treatment, the paper offers a general solution for the deformations of a structure due to the displacement of its joints, and relationships are established between the deformations and the forces they produce.

Analysis of cylindrical tanks with flat bases by moment distribution methods

Amin Ghali, *The Structural Engineer* (London), V. 36, No. 5, May 1958, pp. 165-176

Author's Summary

Moment distribution method is used to find moments and ring tension in walls and bases of the following types of cylindrical tank: (a) cylindrical tank on a rigid flat foundation, (b) cylindrical tank with flat base supported on a cylindrical shaft of smaller diameter.

For the first type a trial and error method is used to determine the width of the ring-

shaped part of the base which will lift up from the foundation surface.

In the second type the variation of the thickness in the overhanging part of the base is taken into account. Graphs are presented in the appendix to facilitate the solution.

The design of each type is illustrated by a numerical example.

Prestressed and reinforced concrete work at Ansells Brewery

J. E. C. Farebrother, *The Structural Engineer* (London), V. 35, No. 9, Sept. 1957, pp. 329-339
Reviewed by C. P. Siess

Describes layout and design details for three structures of complicated layout and carrying relatively heavy loads from machinery and vessels.

A moment-distribution method for Vierendeel bents and girders with inclined chords

Edgar Lightfoot, *Proceedings*, Institution of Civil Engineers (London), V. 10 (Session 1957-58), July 1958, pp. 321-352
Reviewed by Aron L. Mirsky

Author's method is an extension, rational simplification, and generalization of three recent developments: Maugh's "panel method" (1935), the "no-shear" method, and Wood's "degree-of-fixity" method (which reduces difficulties due to slow convergence). Application of method is illustrated by three examples originally worked by other methods.

Flexure of arches and of beams with variable moment of inertia (Flambement des arcs et des poutres à inertie variable)

Roger Guillot, *Le Génie Civil* (Paris), V. 135, No. 10, May 15, 1958, pp. 232-233
Reviewed by Aron L. Mirsky

Using $I = I_o/[1 - k(x/a)^n]$ $(n \geq 2)$ to express the variable moment of inertia, reduces the governing differential equation to the form soluble in confluent hypergeometric form,* for arches of various degrees of redundancy. Convergence of the series solution is shown to be good. Extension to other types of structures (frames, rings, plates) and to lateral bending is mentioned.

*See, for example, Hildebrand, F. B., *Advanced Calculus for Engineers*, Prentice-Hall, 1948-49, pp. 178-185.

Design of grid and diagrid systems on the analogy of design of plates

K. J. Taraporewalla, *The Structural Engineer* (London), V. 36, No. 4, Apr. 1958, pp. 121-128
Reviewed by C. P. Siess

Derives equations of flexure for orthogonally anisotropic plate and presents double-trigonometric series solution for rectangular plates. Grid or crossing-beam system is then analyzed by choosing suitable flexural and torsional stiffnesses for analogous slab. System with rectangular grid of beams inclined to boundaries of plate (diagrid) is solved with aid of Mohr's circle to obtain moments in diagonal beams. Includes examples for both grid and diagrid systems.

Critical loads of tall building frames (Part IV)

R. E. Bowles and W. Merchant, *The Structural Engineer* (London), V. 36, No. 6, June 1958, pp. 187-190
Reviewed by C. P. Siess

Previous papers on this subject appeared in *The Structural Engineer*, July 1954, Mar. 1955, Aug., and Sept. 1956. This paper gives approximate methods for the replacement of a multibay frame by an equivalent single bay frame (of the type considered in a previous paper) for the purpose of calculating the sway critical load. The accuracy of this method is shown by comparison with an accurate solution presented previously.

An improvement of the Myklestad method for flexural-vibration problems

S. Mahalingam, *Journal of the Aero-Space Sciences*, V. 26, No. 1, Jan. 1959, pp. 46-50
Reviewed by Aron L. Mirsky

Myklestad's method* is improved to produce rapid convergence of successive approximations for succeeding higher natural frequencies. Method is based on change of natural frequency consequent on a small (fictitious) change in either the mass of one particle or the elasticity of one segment of the beam, depending on the boundary conditions. Various end conditions are investigated, and a numerical example is given.

**cf.* Bishop, R. E. D., *The Engineer*, Dec. 14 and 21, 1956, pp. 838-840, 874-875; *Proceedings*, Institution of Mechanical Engineers, V. 170, No. 29, 1956, pp. 955-968; "Current Reviews," ACI Journal, Apr. 1958, p. 911 and Jan. 1958, p. 627, respectively.

Foundation beams on elastically yielding soil (Fundamentbalken auf elastisch nachgiebigem Boden)

L. Foppl, *Forschung auf dem Gebiete des Ingenieurwesens* (Düsseldorf), V. 23, No. 6, 1957, pp. 201-232
Reviewed by Aron L. Mirsky

The general integral is set up and a general solution obtained. Special solutions are then obtained for: (1) a long beam with a single centrally-applied load, (2) short beams of various lengths with a single load at the center (the calculated soil pressure distribution is found to change from convex to concave form as the beam length decreases), and (3) beams with two or three concentrated loads symmetrically placed. Results of photoelastic studies are also given and shown to be in good agreement with the analytical results.

Simplified analysis of plane pile grids (Vereinfachte Berechnung ebener Pfahlroste)

Felix Zimirski, *Der Bauingenieur* (Berlin), V. 33, No. 4, Apr. 1958, pp. 149-152; discussion, V. 33, No. 12, Dec. 1958, p. 483
Reviewed by Aron L. Mirsky

Determination of pile forces is simplified by application of graphics, based on energy principles. Two worked examples are given.

On conjugate systems and their use in the construction of influence lines (Sur les systèmes conjugués et leur utilisation dans la construction des lignes d'influence)

Alexandre Gheorghiu, *Annales des Ponts et Chaussées* (Paris), V. 128, No. 6, Nov.-Dec. 1958, pp. 807-861
Reviewed by Aron L. Mirsky

An excellent monograph on conjugate systems. Starting with the basic observation that if the rotation vectors of an elastic system are considered as forces and the displacement vectors as moments, the force system thus defined is in equilibrium, author investigates two particularly important cases, those of plane structures deforming within their plane (e.g., beams with a "medium plane"), and plane structures deforming normal to their plane, shows that each structure of the first type has as a conjugate a structure of the second type, and vice versa (the column analogy being a familiar example), and demonstrates the use of the concept for the construction of influence lines for various highly-indeterminate structural systems.

Restrained concrete arch and its stress regulation (in Croatian-Serbian)

M. Trojanovic, *Naše Gradevinarstvo* (Belgrade), V 12, No. 7, Aug. 1958, pp. 147-153; No. 8, Sept. 1958, pp. 176-185
Reviewed by J. J. Polivka

Part 1 presents basic concepts, purposes, and principles of stress compensations originated by Freyssinet and a retrospective review of later development and use of these important and intricate problems, with reference to current technical literature. Author explains that these problems should be solved by considering the plastic flow of concrete, since the creeping of concrete affects, to a considerable degree, the state of stresses in the arch. It is emphasized that the classical theory of design, based on perfectly elastic properties of materials involved, is to be investigated and improved. Expansion methods and their optimum conditions are thoroughly discussed. Special attention is paid to arches with box sections, thin slabs, and ribs. Limiting values of tensile stresses are proposed, especially in sections using a low percentage of longitudinal reinforcement.

Part 2 describes a method of selecting the most convenient moments in concrete arches. Stress compensations and its effects are compared with typical methods of prestressing. Bresse's expressions (1854) are derived and analyzed. Explanation is given why the procedure on the site is based on calculated deformations, using exact modulus of elasticity, the values of which are to be determined by special methods.

Temperature stresses in concrete in the presence of creep (in Russian)

N. Ya. Panarin, *Nauch. tr. Leningr. inzh.-stroit.*, No. 23, 1956, pp. 43-54
Applied Mechanics Reviews
Jan. 1959 (M. A. Zadoyan)

By applying analytical expressions for the rate of creep and time-variation of the modulus of elasticity, as derived in the work of N. Kh. Arutiunian ("Some problems of the theory of creep," *Gostekhizdat*, 1952), the author, can represent the component stresses in the form of the product of the function of

the coordinates and the time function (damping coefficient of the stress).

A solution is presented for the integral equation on creep by reducing it to a second order nonhomogeneous differential equation with variable coefficients.

Introducing certain simplifications in the expressions, author obtains final calculation formulas for the damping coefficient.

Analysis of an end frame for shell roofs

S. K. Ghosh and B. N. Ghosh, *The Indian Concrete Journal* (Bombay), V. 33, No. 2, Feb. 1959, pp. 42-45

Article is confined to the analysis of end frames supporting a shell roof, in particular a tied arch for a barrel shell roof.

Safety factor in structures in the elastic range without stress-load proportionality

A. Zaslavsky, *Bulletin*, Research Council of Israel (Haifa), V. 6C, No. 2, Section C: Technology, Feb. 1958, pp. 135-142

Nonproportionality between stress and load in the elastic range is illustrated in a number of examples. It is suggested that such structures be designed on the basis of the load producing a critical stress rather than on the conventional basis of allowable stresses.

Ultimate load design of steel and reinforced concrete structures (in Hebrew)

A. Zaslavsky, Midoth, Israel, 4th Edition, 1958, 168 pp.

This book is an outgrowth of postgraduate courses for civil engineers given by the author on ultimate load design of steel and reinforced concrete structures. The design of statically determinate and ordinary indeterminate structures is explained and illustrated in about 80 examples. Tables and some portions of various building codes are included.

Subjects covered in the chapters on reinforced concrete structures include: strength properties, axially compressed columns, rectangular sections in bending, shear reinforcement, twin-reinforced rectangular sections in bending, T-sections in bending, eccentrically compressed rectangular sections, axial and eccentric tension, continuous and noncontinuous two-way slabs, rectangular sections in torsion, and rectangular sections under combined skew bending and axial compression.

Statical analysis of frames (in Polish)

Maciej Gierszewski, (Warsaw), 1955, 463 pp.
Reviewed by J. J. Polivka

A thorough presentation of the analysis of many types of frame structures and discussions of known analytical methods accompanied by numerical examples. The book contains 11 chapters covering the following subjects: (1) characteristic properties and basic principles of frames, with references to principles of Maxwell, Betti, and Rayleigh; (2) method of determining equilibrium of forces occurring in simple and multistory frames, effects of temperature changes, and direct stresses; (3) methods of elastic deformation; (4) combined methods of elastic deformation and equilibrium of forces; (5) method based on equilibrium of bending moments; (6) method of fixed points; (7) examination of various conditions of equilibrium; (8) analytical methods by solving equations; (9) suggestions on how to select the most suitable method; (10) tables for various types of loading of members with constant and variable moments of inertia; (11) bibliography. Book has many valuable features for practical design.

Analysis of cylindrical tank walls

J. D. Davies, *Civil Engineering and Public Works Review* (London), V. 54: No. 631, Jan. 1959, pp. 65-67; No. 632, Feb. 1959, pp. 203-205; No. 633, Mar. 1959, pp. 345-347

Author's Summary

Describes an approximate method of determining the elastic behavior of cylindrical tank walls by a straightforward procedure of computation, whereby, the more complex mathematics required in a fully rigorous solution is dispensed with.

Good approximations are possible for tanks with comparatively small shape factors $K = 12h^4/R^2t^2$ and the method is particularly useful for dealing with partially full tanks and tanks with nonuniform walls.

The amount of repetitive computation is much reduced and solutions are obtained more quickly by using the procedure in conjunction with a chart of correction factors derived by mathematical analysis.

Design of square shafts in plain concrete (in Croatian-Serbian)

K. Doluhanov, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 7, July 1958, pp. 164-168

Reviewed by J. J. Polivka

Under certain exceptional circumstances concrete water tanks, shafts, bins, etc., have to be built without reinforcement, and a design of this type needs special considerations and details. Author presents an analysis based on properties of concrete and recommends a suitable shape for walls and bottom slabs. Nomographs show dimensions for various sizes of these structural types.

On the membrane theory of conoidal shells (Zur Membrantheorie der Konoidschalen)

Mircea, Soare, *Der Bauingenieur* (Berlin), V. 33, No. 7, July 1958, pp. 256-265 (an errata appears in Feb. 1959 issue, p. 76)

Reviewed by Aron L. Mirsky

Starting from fundamentals, author derives the basic expressions, investigates various special types of conoids and various loadings, and obtains design formulas for conoids of three shapes (parabolic, catenary, and circular). Two numerical examples are given, and there is a brief but good list of references, all European.

Materials

Void content of aggregate mixtures (Der Hohlraumgehalt von Korngemischen)

Fritz Schwanda, *Beton* (Düsseldorf), V. 9, No. 1, Jan. 1959, pp. 12-17

Author's Summary

Presented is a method for determining the void volume of aggregate mixtures by calculation if the grading curve is known and the type of aggregate is classified numerically. It is shown that two classifications are necessary which easily are determined by tests. The method renders it feasible to find for each type of aggregate (i. e., smooth, rounded particles or rough, angular, elongated ones) the grading containing the smallest void volume which can be attained with the material given, and to predict the influence of any deviation from that gradation.

Paper starts with the treatment of mixtures of a fine aggregate with a rough surface then investigates the influence of a decreasing ratio of particle sizes. By repeated application of the results gained, a formula is found for the void volume for any number of particle sizes used for the mixture.

The method is applied to the determination of the most advantageous grading of concrete aggregates with respect to workability, cement demand, and economic utilization of the aggregate available. Finally the results derived by calculation are compared to the results of experience gained since Fuller's pioneer investigations.

The method is applicable also to other branches of science, where particle mixtures play a part, i. e., soil mechanics, flow of ground water, and preparation of ceramic materials.

Light cellular concrete—contemporary building material (in Croatian-Serbian)

M. Pajevic, *Naše Gradevinarstvo* (Belgrade), V. 13, No. 1, Jan. 1959, pp. 25-31

Reviewed by J. J. Polivka

Use of this material is steadily growing due to good thermal insulation and rapid construction. Important conditions for further development are discussed and recommended such as serious preparatory work, organized research, and intimate collaboration among industry, engineers, and builders. Research work at Polish testing center and Yugoslav Institute of Testing Materials is thoroughly described, as well as mechanical equipment for mass production.

Structure of aqueous suspensions of pure and mixed cements (in French)

A. Steopoe, *Revue des Matériaux* (Paris), No. 508, Jan. 1958, pp. 1-9

Reviewed by Phillip L. Melville

A study of structures found in sedimentation from aqueous suspension of cements. Effect of temperature and additives is covered. Fully settled cement has a latticed structure which prevents good compaction. Addition of active minerals cause swelling while inert materials form more compact sediments. Additives also control the time of set by chemical action on the ions Ca_2. Set cement has a network of hairline cracks, cause of poor durability which is extended by crystalline silica and reduced by lime adjunctions.

Determination of mix proportions of mortar and concrete in the presence of limestone (Bestimmungen des Mischungsverhältnisses von Mörtel und Beton bei Anwesenheit von Kalkstein)

K. H. Schmidt, *Zement-Kalk-Gips* (Wiesbaden), V. 11, No. 11, Nov. 1958, pp. 491-493
Reviewed by H. H. Werner

The determination of limestone content is based on the difference in radioactivity of CO_2 in the limestone and in the binding agent. CO_2 in limestone is practically non-radioactive while the binding agent contains the normal radioactive atmospheric CO_2.

Some notes on the characteristics of ferro-cement

Lyal D. G. Collen and R. W. Kirwan, *Civil Engineering and Public Works Review* (London), V. 54, No. 632, Feb. 1959, pp. 195-196

While much has been published about the work of Pier Luigi Nervi, little has been written in detail in English about the mechanical properties of his material called ferro-cement. The authors thinking that conditions in the building industries of Italy and Ireland were not dissimilar, felt that the material has possibilities in the Irish economy and conducted a series of preliminary tests.

The tests were aimed at establishing the properties of ferro-cement in bending and shear and sought to establish a value for E, and an optimum percentage of steel consistent with economy. Although the tests were exploratory, they do demonstrate the possibilities of ferro-cement as a material capable of flexible architectural treatment. The authors indicate that further research is indicated, particularly in the more effective use of the steel.

Variations in the strength of portland cement

P. J. F. Wright, *Magazine of Concrete Research* (London), V. 10, No. 30, Nov. 1958, pp. 123-132
Author's Summary

Data on the variation in compressive strength of ordinary portland cement have been collected from tests carried out by a number of investigators. The tests were made on mortar cubes made and tested as described in BS (British Standard) 12:1947 or on 4-in. concrete cubes made and tested as described in BS 1881:1952.

Bond characteristics of cold-twisted square steel reinforcing bars

H. J. Brettle, *Constructional Review* (Sydney), V. 32, No. 3, Mar. 1959, pp. 29-32
Author's Summary

The average bond stresses obtained from pullout tests on vertically and horizontally embedded cold-twisted bars are given. The method of concrete compaction has a marked effect on the average bond stress obtained from vertically embedded bar specimens.

Release of alkalies by sands and admixtures in portland cement mortars

W. C. Hansen, *ASTM Bulletin*, No. 236, Feb. 1959, pp. 35-38

A test in which $1 \times 1 \times 11\frac{1}{4}$-in. mortar bars, after being leached with water, are used to study the ability of admixtures and aggregates to release alkalies to the liquid phase of a hardened cement paste; and also the ability of these admixtures to combine with alkalies in a mortar. It appears that certain admixtures and aggregates might release alkalies in sufficient quantities to produce expansions in concrete if the aggregate also contains minerals which react with alkalies.

Storing cement

F. S. Hespe, *Constructional Review* (Sydney), V. 32 No. 1, Jan. 1959, pp. 31-36

Reviews the deterioration of stored cement caused by absorption of moisture either directly or from the atmosphere.

The effects of storage time on 28-day compressive strength (and early strength) and kinds of storage (warehouse and site) are discussed.

This paper generally confines its attention to storage of 2 years and less.

Discussion meeting held in Vienna concerning "Cement Hydration" (Diskussionstagung "Ueber die Hydratation des Zementes" in Wien)

Zement und Beton (Vienna), No. 13, Aug. 1958, pp. 17-19
Reviewed by Fritz Kramrisch

Report and excerpts of several lectures held at the meeting, by Czernin, zur Strassen, Schwiete, Keil, Moore, Gille, and others about the present status of knowledge regarding the physical and chemical consistency of the hydration products of cement.

Effect of warm storage conditions on the strength of concrete made with high-alumina cement

ADAM M. NEVILLE, *Proceedings*, Institution of Civil Engineers (London), V. 10 (session 1957-58), June 1958, pp. 185-192

Reviewed by ARON L. MIRSKY

Report of investigation into behavior of concrete cubes made with high-alumina cement and stored in humid atmospheres between 25 and 40 C both continuously and intermittently. Loss of strength was found to occur in all cases, even at temperatures little above normal; the higher the temperature and humidity, and the weaker the concrete, the larger the loss. Intermittent periods of storage at colder temperatures did not result in a full recovery of strength, although the over-all loss of strength is less marked.

Changing views on the hardening of cement (Die Erhärtungstheorie im Wandel der Zeiten)

H. KUHL, *Zement-Kalk-Gips* (Wiesbaden), V. 11, No. 12, Dec. 1958, pp. 525-529

Reviewed by H. H. WERNER

Radiographic and electron-microscopic investigations have brought Le Chatelier's "crystal theory" and Michaeli's "colloidal theory" closer together, but have not solved the problem of the hardening cement.

On the sedimentation of cement suspensions (Ueber Sedimentierung von Zement-Suspensionen)

A. STEOPOE, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 2, Feb. 1959, pp. 67-69

Reviewed by H. H. WERNER

Sedimentation influences are analysed as: composition, temperature, admixtures, electrolytes, etc. Sedimentation may be associated with a change in chemical composition.

Determination of sulfate in cements (Sulfatbestimmung in Zementen)

M. WALLRAF, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 2, Feb. 1959, pp. 55-65

Reviewed by H. H. WERNER

Analysis of sulfate content in cement by means of turbidity measurements and titration with EDTA is described. Instrumentation, procedures, sensitiveness, accuracy, and required time are given, with extensive bibliography on the subject matter.

Cold twisted steel as reinforcement in axially compressed concrete columns (in Hebrew)

A. ZASLAVSKY, *Journal*, Association of Engineers and Architects in Israel, V. 16, No. 1, Jan. 1958, pp. 11-13

AUTHOR'S SUMMARY

In contradistinction to cold stretched steel, the yield point of which is raised mainly in tension, cold twisted steel has a higher yield point in compression as well as in tension.

Axially compressed concrete columns reinforced with twisted steel are therefore believed stronger (and tougher) than columns reinforced with ordinary mild steel. The use of cold worked steel according to different codes of practice is explained.

Effectiveness of concrete admixtures in controlling transmission of moisture through slabs-on-ground

Publication No. 596, Building Research Institute, Oct. 1958, 33 pp., $1.50 (available from Publications Office, National Academy of Sciences—National Research Council, Washington 25, D. C.)

A study of effectiveness of concrete admixtures in controlling the transmission of moisture through slabs-on-ground conducted for the Federal Housing Administration by the Building Research Advisory Board.

The committee did not find adequate data to demonstrate the effectiveness of any admixture to reduce the transmission of moisture through concrete slabs-on-ground in a manner sufficient to replace either a vapor barrier or granular base, or both under conditions where such protection would be needed.

The report also presents a test method for determining the effectiveness of any concrete admixture to protect slabs-on-ground against excessive ground water or water vapor transmission.

Contribution to the study of pozzolan cements (in Croatian-Serbian)

V. TUFEGDZIC, *Naše Graderinarstvo* (Belgrade), V. 12, No. 5, June 1958, pp. 99-105

Reviewed by J. J. POLIVKA

Yugoslavia has volcanic ashes of good pozzolanic properties. At present their use is based on the results of thorough investigations and tests made at the Technical University in Belgrade. The results of these tests are presented in this paper and are compared with pozzolan cements of other countries.

Evaluation and selection of aggregates for concrete construction

G. C. Price, *Roads and Engineering Construction* (Toronto), V. 96, No. 10, Oct. 1958, pp. 88, 90, 92, 94
Highway Research Abstracts Apr. 1959

Significant characteristics of concrete aggregates in Manitoba, Saskatchewan, and Alberta, Canada, are described, followed by research conducted and techniques used by the Prairie Farm Rehabilitation Administration in aggregate evaluation.

Evaluation of aggregates from many sources over the past 10 years has been conducted as a continuing practical research program. Methods such as freeze-thaw testing have been developed and modified in an attempt to evaluate aggregates in light of the use for which they are intended. Special emphasis is placed on visual appraisal of aggregates. Petrographic analysis is considered an important phase of aggregate evaluation. The research is also carried on in the field where detailed data on construction of materials and methods are recorded and a case history maintained of the structures in post-construction years.

Analysis of factors affecting the method of determining the strength of cement with particular reference to a proposed transverse and a modified cube compression test

M. F. Kaplan, *Bulletin* No. 16, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria, Mar. 1958, pp. 47-66
Author's Summary

The problems involved in the preparation of test specimens for the purpose of determining the strength quality of portland cement are discussed. The question whether sand should be mixed with the cement is considered as also the type and proportions of sand to be used, the quantity of water to be added, and methods of mixing and compaction. The size of specimens and systems of loading in a transverse test are discussed with reference to statistical theories regarding the stresses present at failure of brittle materials.

Recommendations are made in regard to the above questions for the purpose of a proposed transverse test and a modified cube compression test for cement strength.

Pozzolan cements (in Croatian-Serbian)

V. Tufegdzic, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 3, Mar. 1958, pp. 54-56
Reviewed by J. J. Polivka

Definitions and properties of natural and artificial pozzolan cements and their practical uses in concrete structures are discussed. Referring to the progress in hydrated lime and portland cement, author emphasizes the potential of natural pozzolans as admixtures for concrete exposed to corrosion. Describes pozzolanic properties of volcanic ashes found in Yugoslavia.

Corrosion of reinforcement in reinforced concrete (Corrosion des armatures dans le béton armé)

Supplement to *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), V. 11, No. 130, Series 40, Oct. 1958, pp. 1083-1086
Author's Summary

Some studies reported by Laboratoire Public d'Essais et d'Études á Casablanca.

Corrosion of steel in a prefabricated joist floor for a private dwelling was attributed to an excessive proportion of chlorides which makes the concrete negative and, for an appreciable potential difference, causes the corrosion of the steel.

The floor of a reinforced concrete bridge was similar subject to corrosion due to excessive chlorides contained in aggregate extracted from dried-out river beds which contained a considerable quantity of soluble salts.

Lastly, corrosion of reinforcing steel for a large bridge was caused by the presence of water which had accumulated in air pockets and which was sealed in by a waterproofing treatment.

On the question of lightweight aggregate (Zur Frage des Blähtons)

F. Kaempfe, *Zement-Kalk-Gips* (Wiesbaden), V. 11, No. 10, Oct. 1958, pp. 437-441
Reviewed by H. H. Werner

Report on experiences with various raw materials for development of a lightweight aggregate plant in Australia. Raw material must show plate or scale like parallel structure of proper grain size. Heat treatment of only a few minutes is recommended and different mechanisms of expansion is distinguished as: perlite and vermiculite type of expansion.

Pavements

Calculation of cement concrete road slabs for destructive loading (in Russian)

I. A. Mednikov, *Trudi Moskovskogo, Matematicheskogo Obechchestvo* (Moscow), No. 16, 1955, pp. 66-78
Applied Mechanics Reviews Mar. 1959 (A. R. Rzhanitsyn)

A theoretical analysis of the load-carrying capacity of an unlimited concrete slab resting on a continuous foundation and uniformly loaded over a small circular area. The solution is founded on a number of assumptions concerning the shape of the surface and the annular crack, distribution of principal tensile stresses over slab, and the conditions for the mechanical resistance of the concrete. For the determination of the failure load, the conditions are set up the for limiting equilibrium of the part of the slab within the circular crack. Formulas and numerical examples are given.

The application of the method of limiting equilibrium is contradictory to accepted assumptions concerning the brittle character of fracture of the material working in tension.

Suitability of truck-mixed concrete for pavements

Albert G. Timms, *Concrete Construction*, V. 4: No. 1, Jan. 1959, pp. 6-8; No. 2, Feb. 1959, pp. 1-4; No. 3, Mar. 1959, pp. 6-9

The use of truck mixers to make concrete for pavement construction has long been a subject of debate among contractors and people responsible for the quality of pavements. This series of articles explores recent literature and reports of experience with truck-mixed concrete.

The objections most commonly raised by federal, state, and municipal organizations to the use of truck mixers for making pavement concrete were examined in Part 1. In the second part, the necessary steps and precautions required to produce uniform concrete were described. Subjects covered in the second installment included plant facilities, control of materials, method of loading, speeds for mixing and agitating, mixing time, and discharge time.

The third article dealt with approved discharge procedures and special problems of hot and cold weather concreting. A summary of recommended practices was also included.

Recommended procedures for determination of relative merit of field curing methods for portland cement concrete pavement

HRB Committee on Curing of Concrete, *Circular* No. 381, Highway Research Correlation Service, Highway Research Board, Mar. 1959, 6 pp.

This procedure is intended to provide data on the effects of curing methods as applied under conditions of normal construction operations on (1) variation in the temperature of the concrete in the pavement during the curing period, (2) the strength of the concrete, and (3) the resistance of the surface of the pavement to abrasion, frost action, or other disintegrating agents.

New runway at Marseilles-Marignane airport (in French)

P. Marie and H. Royer, *Travaux* (Paris), No. 294, Apr. 1959, pp. 199-225

The 2400 x 45-m runway consists of a 0.15 m thick layer of carefully compacted all-in material, 0.10 m of macadam semigrouted with bitumen emulsion, a 0.01 m thick carpet of hot-mix bitumenized sand, and a 0.25-0.28 m thick concrete slab constructed without an interposed layer of sand, so as to increase the friction and even to achieve a certain amount of bond, with a view to dispensing with expansion joints.

Of particular interest are the appendices which deal with the spacing of contraction flexure joints, the movements of concrete runways, and the resistance to buckling of concrete pavements due to expansion.

Performance of granular subbases under concrete

Bulletin No. 202, Highway Research Board, Dec. 1958, 79 pp., $1.60

A collection of three papers.

"Concrete Pavement Subbase Study in Ohio," by L. D. Childs and F. E. Behn, reports the observed condition of an experimental highway constructed in 1952. Several conclusions and indications are given.

"Performance of Subbases for Concrete Pavements Under Repetitive Loading," by B. E. Colley and W. J. Nowlen, reports laboratory data on the performance of subbases under 50,000 repetitions of load. The type and gradation of subbase materials, and the placement condition relative to density and moisture content were evaluated.

"Effect of Base Courts Gradation on Results of Laboratory Pumping Tests," by W. P. Chamberlin and E. J. Yoder, reports results of a laboratory study to investigate the performance of a variety of base course samples with different gradations.

Use of concrete for patching spalled areas in concrete roads

R. H. H. Kirkham, and G. E. Higgins, *The Surveyor* (London), V. 118, No. 3486, Feb. 14, 1959, pp. 151-154

Areas that have spalled in the middle of concrete slabs can be patched successfully with concrete provided that the surrounding concrete is sound, that a vertical edge is formed, and that the area is cleaned thoroughly. Patches only ¼ in. thick are satisfactory after 4 years on roads carrying light traffic. Two methods were used: pneumatic gun and filling by hand.

When the methods used successfully for areas in the middle of slabs were tried near joints on public highways, however, they were not as satisfactory.

Trials with mortar containing polyvinyl acetate, which it was suggested would increase the adhesion of the patch or enable areas to be patched without the labor and expense of breaking out, were not successful.

Precast Concrete

Progress in the development of assembled concrete structures (in Russian)

N. M. Kurek and S. N. Scerbakov, *Novaja Teknika i Peredovoi Opyt w Stroitel'stve* (Moscow), No. 11, 1957, pp. 6-12

Reviewed by N. G. Zoldners

Discusses recent achievements in the development of prefabricated reinforced concrete members for assembled structures in USSR.

This type of construction was widely propagated in the first years after the war as a substitute for wood and steel building materials. Floors and roofs made of precast concrete slabs were followed by a series of industrial buildings made completely of prefabricated reinforced concrete structural members. At the same time the rapid growth of the steel industry discouraged wider application of precast concrete members in the building trade.

The decisive push for general acceptance and mass production of precast structural concrete was given by the government decree issued in August 1954. The decree ordered erection of new shops and casting plants to produce prefabricated concrete structural members; provisions for the study of all necessary plant equipment and machinery; definite volume of production for each plant per year; application of the principle of assembling prefabricated concrete members where possible in the designs of new buildings.

This decree had tremendous impact on the technical education in the building trade. The new field opened possibilities to develop advanced methods in building technique leading to savings in material and manpower.

Brief descriptions of the following types of assembled reinforced concrete structures and members are given:

Simple prestressed beams and composite girders with spans of 12, 18, and 24 m made of precast single blocks each 3 or 6 m long. Composite prestressed trusses covering spans of 18, 24, 30, and 36 m. Prestressed roof and floor slabs, concrete shell structures.

The volume of precast concrete produced in the USSR in 1950 was 1.3 million cu m, this was increased in 1958 to 17.0 million cu m. The goal for 1960 is 28.0 million cu m, of this 25.0 percent will be prestressed as compared with 4.7 percent in 1957.

Vibration used in the mass production of concrete elements (La vibration: Utilisée dans la fabrication en série d'éléments en béton)

Jean-Claude Borel, *Cahier* No. 285, Centre Scientifique et Technique du Bâtiment (Paris), Dec. 1958, 38 pp.

Author's Summary

A review of familiar notions on vibration is followed by an account of a series of trials conducted at the Champssur-Marne Experimental Station, in which particular parameters in concrete mix proportions (water, cement, aggregate) and in vibration (amplitude, frequency) were varied.

The author advances criteria for the appraisal of fresh concrete, capable of immediate application without waiting for the concrete to set, thus enabling a line production to be checked step by step. The trials reveal the sensitivity of the results of vibration to vari-

ations in certain parameters. Some of the trials also demonstrate how to match concrete to a given vibration frequency by increasing the number of certain granules in the aggregate.

Appendices examine the structure of a large mass of concrete during and after vibration and describe the plant used.

Concrete block for walls (Bloques de hormigón para muros)

J. M. Eymar and J. Puente, *Bulletin* No. 194, Instituto Técnico de la Construcción y del Cemento (Madrid), 1948, 97 pp.

Authors' Summary

The various types of block employed in different countries are described, also specifications, results of tests, and details of methods of construction applicable to them. Other chapters deal with machinery and other equipment for the manufacture, curing, proportioning, etc., of the block, as well as types of aggregate most frequently used.

Quality control in factory-made precast concrete

A. R. Cusens and J. H. Wettern, *Civil Engineering and Public Works Review* (London), V. 54, No. 634, Apr. 1959, pp. 465-468

Authors' Summary

Deals with some of the problems peculiar to factory produced concrete. Besides touching on the principal factors involved, viz., materials, workmanship, and testing requirements, an analysis is made of a number of cube results taken over several years. Conclusions are drawn as to the suitability of different batching methods, and comment is made on principles of fixing target strengths for cube results, which guide the mix proportioning and provide an incentive for the workers.

Problems of prefabricated family houses (Problémy montovaných rodinných domov)

S. Francu, *Stavebnícky časopis* (Bratislava), V. 6, No. 5, pp. 257-288

Reviewed by J. Stork

Discussion of an economical layout of a simple one floor family houses for urban as well as country use, and the variability of the layout. The construction is of precast lightweight reinforced concrete slabs; expanded clay (keramzite) used as aggregate. Modular grid is 180 x 180 cm (6 x 6 ft). Attention is given to the construction of cellarless houses.

New method of manufacturing pipes using reinforced concrete (in Serbian)

B. Nesovic, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 10, Oct. 1958, pp. 224-228

Reviewed by J. J. Polivka

Importance of pressure pipes is discussed. Advantages and disadvantages of both metal and reinforced concrete pipes is described on the basis of tests. Author suggests various improvements, especially lining with polyvinyl, steel sheets, and asbestos cement.

Precast concrete elements for dwelling construction (Betonfertigteile für den Wohnungsbau)

Kurt Zielinski, *Die Bauzeitung* (Stuttgart), V. 63, No. 4, Apr. 1958, pp. 173-175; No. 5, May 1958, pp. 240-243; No. 6, June 1958, pp. 288, 290, 292

Reviewed by Aron L. Mirsky

Sketches and tabulations, with discussion, of standardized elements (lintels, cellar light shafts, stairs, and cornices) developed in Schleswig-Holstein. (Abstracted from "Construction in Schleswig-Holstein," *Bulletin* No. 23, Arbeitsgemeinschaft für zeitgemässes Bauen, e.V., Kiel).

Actual development of prefabricated concrete structures in Poland (in Rumanian)

Dinu Morariu and Lazar Abramovici, *Revista Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 7, July 1958, Part 1, pp. 379-391

Reviewed by J. J. Polivka

A description of materials used and their characteristic properties, types of structural elements (hollow floor panels, I-and T-shaped joists and beams, stair flights, circular and undulated thin shells, frames, long-span trusses up to 100 ft), and various precast and prestressed concrete structures.

Recent example of heavy precast concrete construction in a mill building at Northfleet

A. E. Beer and L. A. Macer, *The Structural Engineer* (London), V. 36, No. 6, June 1958, pp. 191-196

Reviewed by C. P. Siess

Project involves three buildings for paper mill designed for heavy loads. Both prestressed and precast construction used in

conjunction with cast-in-place concrete. Secondary beams connected to main girders by steel brackets attached to the girders by bolts and resisting shear solely by friction.

Prefabricated apartment house construction (in Dutch)

MAZURE, *Cement* (Amsterdam), No. 21-22, 1958, pp. 865-868
T' HART, *Cement* (Amsterdam), No. 23-24, 1958, pp. 933-940

Reviewed by JOHN W. T. VAN ERP

Mass production methods call for (1) mechanization, (2) standardization, (3) prefabrication. Various systems are analyzed particularly those using large elements of up to 3 tons, erected by gantry crane. Amount of skilled labor at the job is minimized, of which there is still a critical shortage. Large elements are usually a complete wall or floor unit of a room, and those of one apartment are not in direct contact with those of the next, making for good sound insulation. Elements are tied together after erection by placing concrete in the joints, for which no formwork is required.

Prestressed Concrete

Masts prestressed by the "tube-pull" method (Spannbeton-Maste nach dem "Rohrzug-Verfahren")

L. FORKERT *Zement und Beton* (Vienna), No. 13, Aug. 1958, pp. 12-17

Reviewed by FRITZ KRAMRISCH

Description of a manufacturing process for masts for street lighting, transmission lines, etc., employing the post-tensioning system to use heavier bars for better grounding, and greater safety against rusting. The bars are laid in smooth tubes that are pulled out as soon as the concrete has been compacted sufficiently to maintain its form, but before it has set. Bars are bent to U-shape and do not have to extend for the full length, they are anchored in the upper portion of the mast and are stressed from the lower end. Grouting is effective due to lack of tubes and, vibration of the entire mast during the grouting operation. This system does not require final end anchorage. After the grout has hardened, the jacks are removed and the projecting bar ends are cut off.

Cracking in prestressed concrete

JAMES R. LIBBY, *Modern Concrete*, V. 22, No. 11, Mar. 1959, pp. 48-53

One of the basic problems in the prestressing industry is cracking. This discussion deals with the conditions that affect cracking: (1) external loads; (2) physical, chemical changes; (3) poor workmanship and construction methods; and (4) poor detailing practice.

Relative merits of plain and deformed wires in prestressed concrete beams under static and repeated loading

STEPHEN C. C. BATE, *Proceedings*, Institution of Civil Engineers (London), V. 10 (Session 1957-58), Aug. 1958, pp. 473-502

Reviewed by ARON L. MIRSKY

Presents results of tests on pretensioned beams made with 12 types of wires, including indented, crimped, and rusted. While ultimate strengths under static loading were higher with some types of deformed wires, no beneficial results were obtained from their use in beams under repeated loading. All types tested were able to sustain a sufficiently large range of repeated stress to ensure an adequate factor of safety against fatigue failure of the prestressed concrete beams. For discussion of this paper, see *Proceedings*, Feb. 1959, pp. 223-225.

Design charts for prestressed concrete beams forming a deck

REGINALD G. ROBERTSON, *Proceedings*, Institution of Civil Engineers (London), V. 10 (Session 1957-58), May 1958, pp. 19-38; discussion Dec. 1958, pp. 499-500

Reviewed by ARON L. MIRSKY

Scope of design charts for determining economic proportions of prestressed beams, previously published by author* is extended to (1) beams whose top flanges form the deck, (2) composite beams with top flanges cast, either partially or wholly, in situ, and (3) composite slabs formed of inverted T-beams with in-situ filling. Equations, design charts, and worked examples are given, and there is a brief discussion of the effect of differential shrinkage and creep on stresses in composite-type beams.

*_Proceedings_, Institution of Civil Engineers, Part 3: V. 3, No. 1, Apr. 1954, pp. 242-297; V. 4, No. 1, Apr. 1955, pp. 155-174; V. 5, No. 1, Apr. 1956, pp. 189-195; "Current Reviews," ACI JOURNAL, V. 27, No. 2, Oct. 1954 (_Proceedings_ V. 51) p. 208; V. 28, No. 5, Nov. 1955, (_Proceedings_ V. 52) p. 393; V. 29, No. 4, Oct. 1956, (_Proceedings_ V. 53) p. 428.

Huge prestressed sandwich panels span 61 ft

Concrete Products, V. 62, No. 3, Mar. 1959, pp. 38-39

Reports on the use of prestressed panels, 61 ft long, for the construction of a transformer station. The panels incorporate a double-T cross section with a thick Fiberglas insulating material centered in the slab. The panels are cured with hot oil and as a secondary technique, a curing compound is applied.

Construction of a prestressed concrete trestle for hydromechanic transportation (In Rumanian)

ST. ANGELESCU and E. BAICULESCU, *Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 7, July 1958, pp. 355-360

Reviewed by J. J. POLIVKA

V-shaped members 8 in. wide at the bottom, increasing to 16 in. at the top, and 2½ in. thick, are assembled from precast sections 42 in. long, and prestressed in a way similar to hollow blocks, thus forming channels spanning 49 ft-6 in. These trestle beams are supported by I-shaped piers, 32 x 36 in., with 32-in. flanges increasing in thickness from 2¾ to 4 in. The webs are 4-in. thick, 20-in. high pier blocks, with vertical cavities, eight on each face, in which two cables, ⅞ in. dia. are inserted (altogether 16 cables) and prestressed to 119 psi, to approximately 70 percent of the ultimate strength. Average height of the piers is approximately 26 ft. Inclined walls of the channel beams have six continuous cavities on each side, in which cables are inserted and prestressed, four in the two lower ones, and one at the top. These cables are raised toward the supporting piers.

Design charts for prestressed concrete continuous beams of uniform section

REGINALD G. ROBERTSON, *Proceedings*, Institution of Civil Engineers (London), V. 10 (Session 1957-58), May 1958, pp. 39-58

Reviewed by ARON L. MIRSKY

Method for determining economic span ratios and depth for three-span continuous beams of uniform I-section subjected to a moving uniform live load and/or a point load, and design charts based thereon, are developed, and their use is illustrated by worked examples. The effect of various factors, such as haunches and large dead load, is investigated and formulas to take these into account are developed and their use illustrated. A design chart for two-span continuous beams is also developed. Designs obtained by author's method are compared to previous designs for haunched continuous beams and for simply supported beams and shown to be more economical.

Impact resistance on prestressed concrete masts

P. W. ABELES, *Publication*, International Association for Bridges and Structural Engineering (Zurich), No. 17, 1957, pp. 1-7

APPLIED MECHANICS REVIEWS Jan. 1959 (Author' summary)

Comparative tests are described which were carried out by British Railways, Eastern Region, 1955, to investigate the impact resistance of steel and prestressed concrete masts. Each type became entirely unserviceable at a direct hit by a 20-ton wagon projected into the mast with a speed of 10 miles per hr, but remained temporarily serviceable at a glancing blow. Further, the suitable shape of prestressed concrete masts is discussed in connection with appropriate design considerations and, finally, 20-years' experience in Austria with nonprestressed spun-concrete masts with high-strength steel reinforcement is reported as satisfactory in spite of unavoidable fine hair cracks.

Prestressed concrete pipes and circular tanks—Parts I-IX

P. W. ABELES, *Civil Engineering* (London), V. 50; No. 593, Nov. 1955, pp. 1235-1238; Dec. 1955, No. 594, pp. 1375-1378; V. 51, Jan. 1956, No. 595, pp. 83-84; Feb. 1956, No. 596, pp. 203-206; Mar. 1956, No. 597, pp. 319-332; May 1956, No. 599, pp. 552-554; June 1956, No. 600, pp. 671-672; Aug. 1956, No. 602, pp. 895-896; Sept. 1956, No. 603, pp. 1008-1010

APPLIED MECHANICS REVIEWS Mar. 1959 (E. Rathgeb)

After a short historic review of the development of prestressed pipes and tanks, author discusses four principal types of prestressed pipe, analyzing stresses due to internal pressure and circumferential prestress and defining factors of safety. Then describes, in some detail, the best-known systems for fabricating prestressed pipe. The last five articles of the series are dedicated to circular prestressed tanks, showing different methods of construction and application of prestress. Special attention is given to factors influencing stress distribution, with tables of coef-

ficients to calculate bending moments and ring forces considering different kinds of bases. Papers give a concise résumé of the subject with a profusion of illustrative sketches and photographs.

Autoclave method for prestressed concrete (in Rumanian)

I. P. Mihai, *Revista Constructiilor si a Materialelor de Constructii* (Bucharest), V. 10, No. 7, July 1958, pp. 389-392

Reviewed by J. J. Polivka

Experiments described are performed at the Institute for Structural Research, Sophia. Experiments present the advantages of steam curing. These are greater resistance against cracking, increase of strength, possibility of using steel wires of lower strength (e.g., 2-3 mm wires having an ultimate strength of 100,000 psi) and greater economy (25-40 percent reduction of cement).

Prestressed and precast concrete floors (in Serbian)

Bogdan Trbojevic, *Naše Gradevinarstvo* (Belgrade), V. 12, No. 7, July 1958, pp. 159-163

Reviewed by J. J. Polivka

Advantages of prefabricating structural elements such as slabs, joists, beams, columns, etc., are described with reference to typical examples and methods used in England, France, Germany, and other countries. Also the combination of concrete with other materials, such as ceramic tiles, is reviewed.

Partially-prestressed concrete structures Tests and practical execution [Teilweise vorgespannte Betonkonstruktionen (Versuche und praktische Ausführung)]

P. W. Abeles, *Der Bauingenieur* (Berlin), V. 33, No. 5, May 1958, pp. 183-190

Reviewed by Aron L. Mirsky

This article continues the discussion of partially prestressed construction begun earlier.* After a brief discussion of crack formation and of static and dynamic tests of this type of construction, author describes in some detail its practical use by the British Railways (Eastern Region) in various railroad structures such as bridges, freight houses, catenary frames, etc.

*Der Bauingenieur, V. 33, No. 3, Mar. 1958, pp. 77-84; "Current Reviews," ACI Journal, V. 30, No. 10, Apr. 1959, p. 1138.

Characteristics of strand prestressing tendons

J. L. Bannister, *The Structural Engineer*, (London), V. 37, No. 3, Mar. 1959, pp. 79-86

To reduce labor costs, strand tendons are being used in England for prestressed concrete. These strands are capable of withstanding a tensile force of from 4 to 150 tons.

Strands of 7 and 19 wires (diameter $\frac{3}{8}$-1 in.) were investigated. They consist of a core wire around which are laid 6 and 18 helical wires, respectively. Mechanical characteristics of pitch, helix angles, area, maximum stress, modulus of elasticity, and efficiency are described and shown. Deformation of helical wires by axial tension and torsion, and deformation of a strand by combined axial load and torsion are mathematically analyzed.

Prestressing of cylindrical shells by means of separate prestressing members (Zur Vorspannung von Kreiszylinder-schalen mit Einzelspanngliedern)

Wolfgang Barthelmes, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 1, Jan. 1959, pp. 14-19

Reviewed by Rudolph Szilard

An analytic investigation of stress distribution in cylindrical shells subject to internal pressure and compressive ring forces produced by separate prestressing cables. Infinitely long cylindrical shells and cylindrical shells with different boundary conditions have been analyzed. Numerical examples illustrate that the stress disturbances caused by prestressing strands placed at usual distances are of negligible order of magnitude. The given method can be used for cylindrical shell roofs prestressed transversely.

Properties of Concrete

Study of the resistance of concretes to frost. The essential characteristics of the methods used in France and abroad

de la Jarrige, *Bulletin*, (RILEM) International Association of Testing and Research Laboratories for Materials and Structures (Paris), No. 41, 1958, pp. 1-15

Whereas the methods used to determine the characteristics of materials are generally standardized, it is striking to observe how varied are the methods employed in the special field of testing frost resistance of

concretes. The essential characteristics of the methods used by 30 laboratories throughout the world are described.

Physical problems in the testing of concrete (Physikalische Fragen der Betonprüfung)

H. Rusch, *Zement-Kalk-Gips* (Wiesbaden), V. 12, No. 1, Jan. 1959, pp. 1-9

Reviewed by H. H. Werner

Preliminary results of basic physical research on the deformation of concrete under load is reported. Time behavior of the stress-strain diagram, volume change, absorption of ultrasonic waves, etc., are investigated. Creep is shown as composed of permanent strain or "delayed plasticity" and delayed elastic strain.

On a number of observations made in the course of frost tests by the conventional method and by a new testing method

P. Lhopitallier, *Bulletin*, (RILEM) International Association of Testing and Research Laboratories for Materials and Structures (Paris), No. 41, 1958, pp. 17-32

A brief report of observations from tests in progress and exploratory tests. One of the main differences between the "standard" method and new method described is that in the standard method the entire test specimen is subjected to the destructive action of alternate freezing and thawing, while in the new method the alternations of freezing and thawing have been induced in a section of the specimen rather than in the complete mass. The direction of the process has also been inverted; that is, thawing of a frozen concrete instead of the freezing of a nonfrozen concrete.

On physical structure and freezing phenomenon of concrete (Betonin fysikaalisesta rakenteesta ja jäätymisilmiöstä)

S. E. Pihlajavaara, *Rakennusinsinööri* (Helsinski), V. 14, No. 12, 1958, pp. 176-181

Author's Summary

Beginning with a discussion on the physical structure and properties of concrete the author follows with the fundamentals of the freezing phenomenon and after that takes a closer look at the properties of concrete and their influences.

Improving the durability of concrete in Maine

Andrew Adams, *Proceedings*, Highway Research Board, V. 37, 1958, pp. 306-328

Results of field inspections and laboratory tests were used to modify design and construction procedures to improve durability of highway bridges in Maine. Over 800 bridges built between 1923 and 1947 were inspected, conditions and construction of laboratory data were recorded on punched cards, and correlations computed between condition of concrete and other variables.

Resistance of concrete and mortar to chemical attack—Progress report on concrete corrosion studies

J. H. P. van Aardt, *Bulletin* No. 13, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria, Mar. 1955, pp. 44-60

Author's Summary

Some laboratory experiments aimed at assessing the resistance of portland cement mortars to aggressive agents are described and some possible measures for reducing their vulnerability are suggested.

Comparison of the compressive and flexural strengths of concrete with and without entrained air

B. W. Shacklock, and P. W. Keene, *Civil Engineering and Public Works Review* (London), V. 54, No. 631, Jan. 1959, pp. 77-80

Authors' Summary

Tests were made to compare the compressive and the flexural strengths, after curing in water or air, of concretes of similar cement content and workability with and without entrained air. Other properties measured were bulk density, ultrasonic pulse velocity, dynamic modulus of elasticity, and change in weight during curing.

Although, in general, the compressive and flexural strengths of the air-entrained concrete were less than those of the plain concrete, the difference in flexural strength of air-dry specimens was negligible. Air-entrained concretes would appear to be less susceptible to the dangers resulting from a lack of artificial curing than plain concretes because the changes in weight during curing and the changes in compressive and flexural strengths due to differences in curing were less for the air-entrained specimens.

Lightweight concrete—plain and reinforced (In Rumanian)

OVIDIU MIRSU, *Scientific Association of Engineers and Technicians of Rumanian Republic* (ASIT), Timisoara, 1957, 252 pp.

Reviewed by J. J. POLIVKA

One of the most complete books on the subject, thoroughly describing and discussing characteristic properties of known types of concrete in which prime considerations are lightness in weight, thermal insulation, economy, and sufficient strength for structural purposes. Material is presented in three parts, each subdivided in chapters referring to specific properties.

Part I discusses the preparation of lightweight aggregates such as blast furnace slag, clay, diatomite, fly ash, shale, slate, and natural materials, such as pumic, scoria, and tuff. Chapter 2 considers the strength of lightweight concrete (with mineral aggregates), e.g., the properties of aggregates and cement, plasticity, vibration, etc.; longitudinal and transverse deformation, moduli of elasticity; and theoretical relations. Chapter 3 describes the properties of plain concrete with porous aggregates, its strength, contraction, permeability, thermal expansion, and insulation. Chapter 4 covers the properties of reinforced concrete. Chapter 5 states the applications of concrete with porous aggregates such as hollow blocks for floors and walls. Chapter 6 reports the various applications of precast structural elements such as panels, joists, beams, etc., their properties, assembly, etc.

Part II refers to macroporous concretes. It gives general properties and advantages, such as specific weight, strength, bond between steel and concrete, contraction, permeability, absorption, thermal conductivity, acoustic insulation, etc. Also covers use of lightweight concrete on construction site, forming, and its advantages.

Part III treats cellular concretes. Covers the materials, aggregates and admixtures, preparation of the paste, effect of reinforcement, and the increase of strength by special curing methods. It further discusses the properties of vacuum concrete such as strength, bond, fire resistance, thermal and acoustic insulation, and the structural elements of their design and fabrication.

Corrosive influences on concrete

P. EBENWEIN, *Schweizer Archiv für Angewandte Wissenschaft and Technik* (Solothurn, Switzerland), V. 24, No. 6, June 1958, pp. 188-191

Reviewed by PATRICK E. MURPHY

Notes that it is through the medium of atmosphere, water, rock, and soil that harmful agents such as sulfates, sulfides, acids, organic substances, etc., are brought in contact with concrete. However if the concrete exhibits a small contact area or is dense their effect is often negligible. It is pointed out that although the worst source of corrosive agents is hydrous solutions, it is also possible that soft water, i.e., water containing practically no chemicals free or in solution can reduce the strength of the concrete by "leaching"—the removal of calcium hydroxide in solution. Protective measures against corrosive influences are discussed. These mainly consist of either sealing the concrete by the use of high quality, dense concrete and painting, or the use of special cements.

Effect of low temperatures on hardening of cement, especially slag cement

L. BLONDIAU, *Revue de Materiaux de Construction et de Travaux Publics* (Paris), 1957, pp. 141-146, 181-188, 203-218, 290-297, 327-331, 364-368; 1958, pp. 15-32. *Journal of Applied Chemistry*, V. 8, Part 9, Sept. 1958, pp. ii-237

HIGHWAY RESEARCH ABSTRACTS Jan. 1959

Extensive tests are reported correlating the strength of concrete mixes with temperature of placing and time of exposure. General conclusions are that slag and supersulfate cements, though more sensitive to low temperature than portland cement, provide just as strong and frost-resistant concrete as does portland cement when used above 3 deg, the enforced delays in removal of shuttering in winter being approximately the same for all three concretes. Troubles in winter concreting arise more from excess water in the mix than from type or quality of cement. All cements should be finely ground and the optimum content for minimum frost sensitivity is 300-350 kg per cu m. The w/c influences strength loss most; vibration before or after placement minimizes adverse effects of low temperature. These effects are limited during the first 14 days to retardation of set and do not produce any permanent disturbance in concrete mixed with the proper amount of water.

Influence of free lime in portland cement on the resistance of concrete to aggressive influences (in Spanish)

F. W. Meier-Grolman, *Report* No. 182, Instituto Técnico de las Construcción y del Cemento (Madrid), 1957, 72 pp; *Building Science Abstracts*, V. 31, No. 4, Apr. 1958, p. 100

Highway Research Abstracts Jan. 1959

Long-term research shows that the hydrate of lime formed by hydrolysis is not affected by the aggressive influences, because it is surrounded by the gel mass formed by topochemical reaction (of the anhydrous calcium silicates), and is thus to a great extent protected. The aggressive modulus of portland cement is formulated as the quotient of the topochemically reacting constituents and those reacting in the liquid phase, and with the aid of this the resistance of cement and concrete can be determined at an early stage from its chemical composition. The critical limiting value for the free CaO content is 0.5 percent of the weight of the cement, in concrete mixes having a cement content of up to 300 kg per cu m—richer mixes are less susceptible owing to the protective effect of their density. By evaluation of the test results it was also possible to ascertain the influence of the C_3A content of portland cement; here the permissible limiting value must not exceed 6 percent.

Structural Research

Recent engineering work at the Building Research Station

The Structural Engineer (London), V. 36, No. 5, May 1958, pp. 180-183

Reviewed by C. P. Siess

Brief review of recent or current structural research at the Building Research Station with references to more complete published reports.

Tests on a model of a reinforced concrete bin (Essais sur un modèle réduit de silo en béton armé)

M. Kellner, Supplement to *Annales* de l'Institut Technique du Bâtiment et des Travaux Publics, (Paris), No. 130, Oct. 1958, pp. 1095-1110

Reviewed by Henri Perrin

Describes tests on a silo cell with truncated cone shaped bin. The model was a half-scale reduction of an existing structure. Compressed air and flat jacks simulated the various loadings to be investigated. Some design criteria are given along with the tests results.

Contribution to the study of rupture in shear of rectangular reinforced concrete and prestressed concrete beams (Contribution à l'etude de la rupture, par effort tranchant, des poutres rectangulaires en béton armé et en béton précontraint)

Kamal N. Ghali, *Cahier* 290, Centre Scientifique et Technique du Bâtiment (Paris), Dec. 1958, pp. 17-28

Author's Summary

The study is concerned with the ultimate shear strength of reinforced and prestressed concrete rectangular beams.

Application of the Mohr-Caquot envelope (known in France as "theory of intrinsic curve") made it possible to develop methods for calculating the orientations of cracks as well as calculating the ultimate strength. If the effect of secondary factors (method of application of loads) is considered, the difference between theoretical and experimental results does not exceed 5 percent.

However, it was judged that rapid formulas for practical calculation of the ultimate strength might be of interest. These formulas were verified against 300 experimental results found in the literature. They permit the calculation of the ultimate shear strength for: (1) reinforced concrete beams with or without compressive reinforcement; (2) reinforced concrete beams with stirrups; and (3) prestressed concrete beams.

Flat slabs supported on columns (in Swedish)

Rolf Baehre, *Nordisk Betong* (Stockholm), V. 2, No. 4, 1958, pp. 359-376

Reviewed by Margaret Corbin

Object of this paper is to determine the deflections of a rectangular flat slab subjected to a concentrated load, and to study the variation in the reactions at the supports of a flat slab carried on a column in the center section of the slab. The investigations are confined to the deflections at the point of application of the concentrated load. The results of calculations are reproduced in the form of tables and diagrams.

Influence of shear span and concrete strength upon the shear resistance of a pretensioned prestressed concrete beam

A. B. Hicks, *Magazine of Concrete Research* (London), V. 10, No. 30, Nov. 1958, pp. 115-122

Author's Summary

The research was planned to establish the various types of failure of a typical pretensioned prestressed concrete I-beam, to establish the limits of the shear span to depth ratio within which each type of failure occurred, and, generally, to investigate the effect of the shear span and concrete strength on the shear resistance of a concrete beam of this type.

Four types of failure were observed; a fifth (bond failure) was carefully avoided. The limits of the shear span to depth ratio within which each type of failure occurred were as follows:

shear distortion	0-1½
diagonal compression	1½-4½ to 5
diagonal tension	4½ to 5-9
flexural	9-∞

Strain gage with metallic mounting-strip as quickly-readied, moisture-immune measuring element for strain measurements on concrete (Dehnungsmeszstreifen mit metallischem Träger als schnell meszbereites, feuchtigkeitsunempfindliches Meszelement für Dehnungsmessungen auf Beton)

Chr. Rohrbach, *Der Bauingenieur* (Berlin), V. 33, No. 7, July 1958, pp. 265-268

Reviewed by Aron L. Mirsky

Describes a gage which uses a standard electric strain gage, is laboratory "prepackaged," easily installed on concrete surfaces in the field, and gives good results. Briefly, standard gage is cemented to a thin metallic strip which is punched with holes along edges to provide a "rivet-like" attachment to concrete in addition to the usual glue layer; leads are attached to cable; and a suitable thin plastic cover is cemented over gage and cable. Technical details of gage characteristics such as fatigue life are given, and brief details of two actual uses are also given. Gage is ready for use 10 min after application to concrete surface, is not affected by moisture in the concrete, and can be removed and reused.

Measurement of the deflection of structures (La mesure de la flèche des ouvrages d'art)

J. Rajade, *Annales des Ponts et Chaussées* (Paris), V. 128, No. 5, Sept.-Oct., 1958, pp. 657-668

Reviewed by Aron L. Mirsky

A discussion of Rabut's (1895) apparatus for the measurement of medium deflections (those between 20 and 0.2mm). Apparatus consists basically of a long wire, one end of which is fixed to the ground or other unyielding reference base and the other end to the structure being investigated, this latter via a spring and a scriber arm. The thermal and other errors involved are discussed, with some reference to experience with tests conducted on Montanges bridge, a reinforced concrete arch of 81 m span, and methods of overcoming them are described.

Three-dimensional investigation of the stresses in the end anchorage blocks of a prestressed concrete gantry beam

S. P. Christodoulides, *The Structural Engineer* (London), V. 35, No. 9, Sept. 1957, pp. 349-356

Author's Summary

The three-dimensional stress distribution in the end anchorages of the prestressed concrete beams, used in the construction of a railway gantry, was investigated by photoelastic methods and electrical resistance and mechanical strain gages.

The results of the photoelastic analysis formed the basis of the design of the anchorage blocks for the beams. and the subsequent strain measurements on one of the beams used in the gantry showed good agreement with this analysis.

The experimental results were compared with the stresses calculated by the standard methods of design which were found to underestimate the stress-concentrations under the prestressing loads.

The way of research

Alfred Pugsley, *The Structural Engineer* (London), V. 35, No. 11, Nov. 1957, pp. 403-407

Reviewed by C. P. Siess

Philosophical discussion of the ways in which structural research is initiated or carried out in the design office, in the development of codes of practice, in the use of new materials, in pioneering projects, and in the universities.

General

Civil engineering handbook

LEONARD CHURCH URQUHART, editor-in-chief, 4th Edition, McGraw-Hill Book Co., New York, 1959, 1174 pp., $17.50

This fourth edition of the well-known handbook reflects many revisions that bring it up to date and in line with current practice. The handbook was written by a number of authorities and gives rules, principles, working methods, and engineering pointers. Like previous editions it aims to present in one volume the fundamentals of the various subdivisions of civil engineering in such a manner that graduates and practicing engineers will find it useful when confronted with problems outside of their specialized field. Sections are devoted to cement and concrete; reinforced concrete; stresses in framed structures; mechanics of materials; soil mechanics and foundations; railway, highway and airport engineering; surveying; hydraulics; steel design; sewage and sewage disposal; and water supply and treatment.

The section on concrete introduces new material on the reproportioning of mixtures for air entrainment and discusses additions and admixtures at greater length and the effect of various substances on concrete. The section on reinforced concrete has been expanded to provide more design aids and expands the information on ultimate strength design and prestressed concrete.

The text on highway engineering has been extensively rewritten, particularly the portions pertaining to interstate highway system requirements and concrete pavement design and construction. The section on airports also includes new material on pavement design. The section on stresses has been completely rewritten with less attention given to railroad structures and more to continuous beams and frames, with the material on the various methods of stress analysis amplified and extended.

What you should know about types of autoclave doors

J. B. MAHER, *Concrete*, V. 67, No. 2, Feb. 1959, pp. 28-32

The types of autoclave doors reviewed are bolted and quick-opening. The quick type door is further classified into three categories; those employing a compressible seal only, a self-sealing type gasket door, and a combination system.

The respective advantages and disadvantages of these doors are considered as well as design, safety devices, operation, and maintenance.

Strength specification of concrete

M. GREGORY, *Commonwealth Engineer* (Melbourne), V. 45, No. 11, June 1958, pp. 57-60, No. 12, July 1958, pp. 61-64; V. 46, No. 1, Aug. 1958, pp. 76-77

Cost reduction can be achieved in concrete works by paying close attention to proper mix proportions and quality control. Arguments for strength specification of concrete and problems arising therein are discussed as well as statistical analysis of strength variations.

Part 2 gives a brief example of the use of statistics in controlling concrete strength by analyzing compressive tests and mix proportions. A draft of several specifications is given.

Part 3 shows how substantial savings can be made by improving quality control and adjusting mix proportions. Two examples are given to exhibit the effect of these ideas.

A critical review of the new SAA concrete code. Part 1—materials, testing, construction

L. BOYD MERCER, *Commonwealth Engineer* (Melbourne), V. 46, No. 4, Nov. 1958, pp. 61-64

A review of the new SAA concrete code. Part 2—design

JOHN W. CONNELL, *Commonwealth Engineer* (Melbourne), V. 46, No. 6, Jan. 1959, pp. 48-50

The Standards Association of Australia (SAA) has issued a totally revised code for concrete in building. The major change is the specification of concrete on a strength basis rather than mix proportions.

Responsibility for quality of the concrete lies with the contractor. He is aided by a criteria of testing procedures and an accompanying statistical evaluation of these tests.

The code also touches on materials and construction practices.

The second part of the discussion reviews the design aspects of the code. This covers, supervision, permissible stresses, flexural members, flat slabs, ribbed and hollow block floors, footings, and earth retaining structures.

ACI News Letter

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A New Service to ACI Members—

Change in Publication of Standards

With this issue of the JOURNAL, a change in publication procedure of standards is effected. The new procedure will provide members with handy individual copies of the standards they use most. The standards, bound separately in easily identified covers, will be sent complimentary, one copy per member, on request. Special order blanks will be published in the News Letter section for ordering these separate copies. Those who use the standards for occasional reference only will still have the proposed standard with any corrections or amendments made to it in the *Proceedings* pages. The new procedure will provide ACI standards in their most convenient form to each member according to his needs.

More JOURNAL space

Now JOURNAL space will not be used for a second printing of a standard; that space will be allotted to additional papers. Instead, a notice of the standard having been accepted will appear along with any amendments or corrections to be made to the proposed version (see pp. 1-3 of this JOURNAL). As before, the complete standard will be published separately in covers and in the *Book of Standards*. In addition to the free copy of the separate standard sent to each member who requests it, copies of the separates and the *Book of Standards* will be available to members at special prices.

In the past a proposed standard or proposed revision to a standard appeared in the *Proceedings* (JOURNAL) prior to a convention as specified in the Bylaws. This procedure will, of course, remain unchanged. Once a standard had been accepted by the convention and ratified by letter ballot, it was again published, in its entirety, in the *Pro-*

ceedings (JOURNAL), in most cases without change. It was additionally published as a separate booklet and included in the *Book of Standards*.

How to order standards

To order the complimentary copy of the standards approved at the 55th annual convention, Feb. 24, 1959, and ratified by letter ballot June 1, 1959, ACI members should fill out the special blank on p. 3. The blank not only constitutes a request for the standards checked, but will be used as a mailing label; therefore, it is important to type or neatly hand letter the required information. Because the printing order will be determined on the basis of requests, they must be received by Sept. 30, 1959.

Photograph courtesy of Blaw Knox Co.

Precast Concrete for Small Structures

Precast slabs and specially designed block reduce the erection time of small farm and commercial buildings, retaining walls and swimming pools. Material costs are approximately the same for a standard block building, but erection costs are about 10 percent less. A dairy barn, 40 x 48 ft, was erected in 4½ hr using this system.

The 15⅝ x 15⅝ x 7⅝-in. block are slotted to receive the standard panels. The 4 x 4-in. slots are cast or cut in various positions in the block according to its place in the structure. Most common are block cast with slots at each end for wall columns and slots in one end and an adjacent side for corner columns. Slots on three and four sides are required for wall columns with an interior wall, and for central columns, respectively.

The columns are set on 24 x 24-in. footings at 12 ft centers. There are no footings under the slab walls so column footings must be large enough to support them. The slots in the block form channels to receive the panels. Panels are 2 ft x 11 ft 3 in. x 3½ in. reinforced with 4 x 4/4 wire mesh. Joints are sealed with caulking compound.

Special panels are installed at window height leaving an opening to receive an extruded aluminum window frame. The 24 x 34-in. frames are installed as the wall goes up, either singly or in pairs.

The system has been patented by Shaffer Block Works, Somerset, Pa.

Michigan members meet at ACI headquarters

Thirty-five ACI members and guests enjoyed a luncheon meeting at the ACI headquarters on June 10. It was the third meeting of the group which is operating on an informal basis before deciding whether to petition the Board of Direction on the formation of a local chapter.

After lunch, Chairman William Krell, introduced Harry Ellsberg, chief structural engineer, Giffels and Rossetti, Architects and Engineers, Detroit, and Chairman of ACI Committee 622, Formwork for Concrete. Mr. Ellsberg traced the background leading to the establishment of the committee and its progress toward the development of a recommended practice for formwork.

After the summer hiatus, meetings will resume in October with a discussion of flat plate construction.

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- ☐ Recommended Practice for Selecting Proportions for Structural Lightweight Concrete (ACI 613A-59)
- ☐ Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-59)

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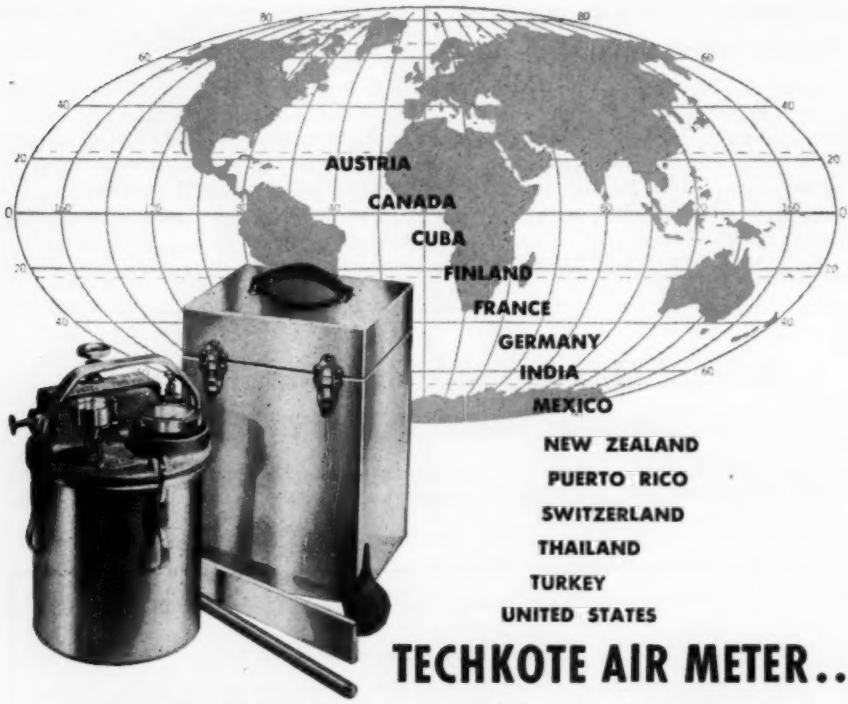

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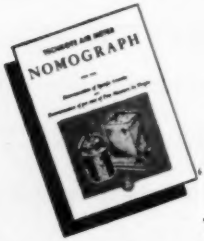

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New Fire Research Laboratory for Canada

THE NEW LABORATORY for fire research of the National Research Council of Canada provides a Canadian laboratory solely devoted to fire research. The laboratory is a part of the building research center of the Division of Building Research, National Research Council, Ottawa.

Fire research has been a part of the Division of Building Research activities since 1950. It was during the revision of Canada's National Building Code in 1953, when many questions arose pertaining to fire regulations concerning buildings, that it was realized that it was essential for Canada to have a laboratory in which fundamental studies could be carried out on many aspects of fire and fire research.

Building description

The building has basically the shape of a large T with offices and laboratories in the front wing, and the large furnace laboratory, with a burn area for model studies, in the rear wing. One-story additions have been erected on either side of the furnace laboratory to provide storage and workshop facilities, garage space, a control room for the operation of the fire resistance furnaces, and additional laboratory facilities for hydraulic testing. The building covers an area of 20,000 sq ft.

The front wing is a two-story structure. Laboratories for small scale experiments and offices for the research staff are on the first floor. Work in the ignition of materials by radiation, possibly small scale model burns, and studies on heat transfer will be carried out in the physics section of the wing. Work in the fire extinguishing field, and the ignition and combustion properties of materials, will be carried out in the chemistry section. Also on the first floor of the front wing are two rooms for pretest conditioning of materials. One of these provides 50 percent relative humidity at 75 F, and the other provides 15 percent relative humidity at 75 F.

Offices, conference room, and library are located on the second floor which extends only over the center section of the front wing.

Testing furnaces

The major portion of the high central part of the building accommodates the furnaces used for the fire resistance testing of wall and floor specimens. At the rear of the furnace laboratory a 40 ft square area, which could be used for future expansion of the furnace laboratory, is devoted to model studies. Models of buildings can be burned in this area in studies of the development of fire.

The wall and the floor furnaces each consist of two parts: a furnace having one open

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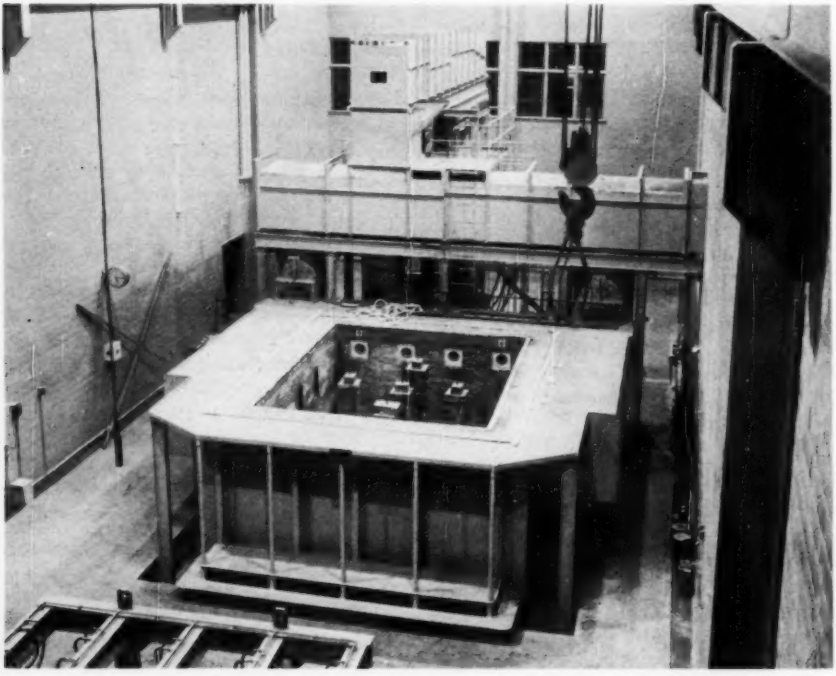
Abstracted from *Building Research in Canada*, V. 2, No. 8, Dec. 1958.

side into which a panel can be fitted, and a means of loading the panel during test. All floors normally carry load and so will, in most cases, be tested under load. Some walls are non-load-bearing and so need not be loaded during tests. Specimens are subjected to fire on one side only, simulating the case of a fire in a single room or space which is being confined by the enclosing construction.

The wall furnace will accommodate a vertical panel up to 14 x 14 ft in the loading frame. Vertical loads can be applied by eight vertical jacks, top and bottom, each capable of exerting a maximum load of 30,000 lb. The furnace, which provides a clear opening for fire exposure of 12 x 12 ft, is mounted on rails and can be run up to the wall panel held in the loading frame. After a test, the furnace can be run back for the application of a hose stream to the hot side of the panel.

The floor furnace consists essentially of a large box having an open top and sunk partially into the laboratory floor. The clear opening of the top of the furnace is 12 x 15 ft. The specimen to be tested must be fitted into a suitable frame. The frame and specimen together can then be lifted to the top of the furnace by a 30-ton building crane and, when lowered into place, forms a closure or lid for the furnace and rests on the furnace walls.

A second steel frame can then be lowered onto the main frame and fastened to it at four loading points. This loading frame carries a series of 30 vertical jacks which can be brought to bear and thus to apply loads vertically downward on the upper side of the test panel while it is exposed to fire



Photos—National Film Board (Canada)

Furnace laboratory in the Division of Building Research fire research building. Floor furnace is in foreground; wall furnace in background. In right foreground is the 30-ton crane used to position panels in the furnaces

on its underside. Each jack has a capacity of 3000 lb so that a simulated floor loading of 500 psf can be attained. Each jack is provided with a tripod head so that its load is applied to three points on the panel, thus providing one loading point on each 2 sq ft of panel area for a good approximation to uniform loading.

Credits

Architect for the structure was Meadowcroft and MacKay, Montreal; resident architects were G. E. Koulomzine and D. Hanlon. Consulting mechanical and electrical engineer was J. P. Keith and Associates, Montreal. General contractor was Thomas Fuller Construction Co., Ottawa, and superintendent was George Hay.

ACI 12th regional meeting—November 2-5—down Mexico way

The technical program for the Institute's 12th regional meeting, scheduled for November 2-5 at the Continental Hilton Hotel, Mexico City, is rapidly taking on an international flavor as the local committee lines up speakers for the technical sessions.

Approximately 20 papers will be presented at five sessions planned to cover quality of concrete; reinforcement; concrete structures; concrete shells; and precasting. The committee is now engaged in selecting papers to assure a worthwhile, highly enlightening technical program. Alberto Dovali Jaime, Universidad Nacional de Mexico, is general chairman of the local planning committee with Adolfo Zeevaert, consulting engineer, serving as secretary, and Antonio Robles, de Cemento Mixcoac, treasurer of the group.

Travel facts

ACI headquarters staff has compiled a few "Mexilaneous" facts for members planning to attend the regional meeting.

Formalities for entering into and departing from Mexico are simple. No passports are required of United States citizens, but proof of citizenship must be carried, such as birth certificate, naturalization papers, passport, voter's card, or affidavit of birth. Canadian citizens must have a Canadian government passport. In addition, everyone must have a Mexico Tourist Card ($3) and a certificate showing smallpox vaccination within the past 3 years.

You can bring personal articles into Mexico free of duty. That includes such items as photographic equipment, 12 rolls of film, four cartons of cigarettes, etc. There are no restrictions on importation or exportation of money except for gold, so don't buy jewelry containing gold coins. United States customs regulations allow you $200 worth of merchandise duty free.

The Mexican peso is worth about 8 cents and a 50 peso note about $4. United States dollars can be changed for pesos at banks, airports, and hotels.

Institute members attending the 12th regional meeting will enjoy an action-packed 3-day technical program highlighted by the colorful, fascinating atmosphere of Mexico City. Few cities in America can compare with Mexico City, full to overflowing with historic, architectural, and scenic wonders.

Precast Concrete Components Lifted and Placed with Richmond Lifting Inserts

Precast wall slab being tilted into place by means of Richmond Lifting Inserts and Lifting Brackets.

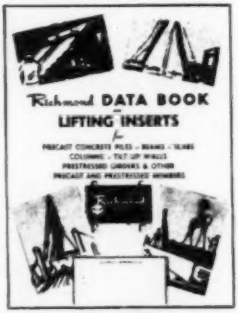
Certified tests by an independent laboratory assure proper strength rating for efficient design and performance in concrete at usable strength

Richmond has developed and tested a complete line of Lifting Inserts and accessories for handling and placing of precast concrete wall slabs, columns, beams, girders, piles, etc.

No matter what type of precast units are involved, Richmond can supply exactly the right type of insert for the specific job.

These units are designed with adequate extra strength and for simple operation in lifting and bracing precast concrete components.

Send for your copy of the Richmond Data Book on Lifting Inserts giving complete technical data, dimensions, working loads and ultimate strengths in various strengths of concrete. At the same time ask for your copy of the latest Richmond Handbook showing the complete line of form tying anchorage and accessory devices for concrete construction backed by 47 years in this field.



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Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 116, Nomenclature

S. H. Westby
Portland Cement Association
Chicago, Ill.

Committee 336, Combined Footings

Ebenezer Vey
Illinois Institute of Technology
Chicago, Ill.

Committee 339, Allowable Stresses in Reinforcement

Richard C. Elstner
Portland Cement Association
Skokie, Ill.

Committee 712, Precast Structural Concrete Design and Construction—Joint ACI-ASCE

Alfred A. Yee
Park & Yee, Ltd.
Honolulu, Hawaii

Committee 716, High Pressure Steam Curing

Edward G. Swenson
National Research Council of Canada
Ottawa, Canada

President Ferguson addresses ACI California chapters

Phil M. Ferguson, president of the American Concrete Institute and professor of civil engineering, University of Texas, Austin, was guest of honor at the ACI Southern California Chapter dinner meeting in Los Angeles, June 10.

On June 17 President Ferguson was guest and principal speaker at the ACI Northern California Chapter dinner meeting in San Francisco.

President Ferguson addressed both meetings on the work and aims of the Institute, with emphasis on its efforts to find logical solutions to structural concrete problems.

As an added feature of the Los Angeles program, a progress report was presented describing the construction of the barrel shell roof of the Union Pacific Freight House in East Los Angeles. This roof comprises 30 barrel shells of 44 ft span, each 117 ft long. Comments and explanations were made by: Eugene E. Hall, resident engineer, Leo Daly Co., Omaha; and W. L. Davis, project superintendent, Diversified Builders, Inc., Paramount, Calif. Robert E. Tobin, Portland Cement Association, served as program chairman.

Cummins appointed ACI technical director

Appointment of Kenneth D. Cummins, formerly associate professor of civil engineering, University of Detroit, as technical director of the American Concrete Institute has been announced by the Institute's Board of Direction.

Mr. Cummins' duties will include field and office work to promote and improve technical relations with other technical societies and organizations and assist member groups in organizing and conducting small local meetings. Upon request, he will assist ACI technical committees in preparation of reports.

Mr. Cummins received his BS in civil engineering from the University of Detroit in 1948 and his MS in structures from the University of Michigan the following year. He has since attended summer courses in highway engineering at Purdue University and at the Institute of Transportation Traffic Engineering, University of California. He is a registered civil engineer in Michigan, Ohio, and Indiana; a registered structural engineer in Illinois; and a registered land surveyor in Michigan.

Professionally, Mr. Cummins has had broad and varied experience. He has engaged in structural steel design; plant engineering (maintenance and new construction); soil boring investigations; aircraft stress analysis; and engineering consultation. He was president of Decker & McGough, Inc., consultants,

from December 1956 to May 1959. While at the University of Detroit, Mr. Cummins was engaged in teaching soil mechanics, highway engineering, reinforced concrete design, and structural steel design.

Mr. Cummins served in World War II with the U. S. Army Corps of Engineers in the European theatre of operations from 1943 until 1945. He later served as a commissioned officer in the Reserve of the United States Public Health Service.

ACI California chapters cooperate in conferences

Cracking and other practical construction problems were discussed by 745 concrete fans at the fourth series of evening meetings held in five California cities during April and May. The previous series were conducted in 1951, 1953, and 1956.

The series was offered by the University of California through its Institute of Transportation and Traffic Engineering and its Engineering and Sciences Extension, with the sponsorship of the Northern and Southern California chapters of ACI and, at Fresno, Fresno State College. Cooperation and support were given by the Portland Cement Association and the Portland Cement Information Bureau.

The meetings were arranged by Bob Glenn, research engineer and extension representative of ITTE. ACI Vice-President Joe W. Kelly, professor of civil engineering, University of California, Berkeley, served as moderator at the five panels.

ACI headquarters draws visitors

The estimated total number of visitors to the Institute headquarters building in Detroit, since it was dedicated last October, increased another 200 during May and June.

In addition to local individuals and groups recently inspecting the structure, the visitors' register shows guests from California, Iowa, Ohio, Canada, and Tokyo.

On May 28, 70 members of the Royal Architectural Institute of Canada visited the building; on June 4, 26 architectural students from the University of Cincinnati inspected the structure.

Yamasaki honored

Minoru Yamasaki was recently honored at a ceremonial dinner highlighting his one-man exhibition at the Architectural League of New York. He is the first architect to be invited by the league to present a one-man show since the late Frank Lloyd Wright 28 years ago.

The show consisted of 30 panels illustrating the chronological sequence of his work. His philosophy and expression of design have won numerous awards. The ACI headquarters building was an award winner for Mr. Yamasaki, taking the top award in the commerce division in the 1957 *Progressive Architecture* composition.

Mr. Yamasaki has been carrying on his work as the principal of the firm of Minoru Yamasaki and Associates, Birmingham, Mich.

Goldes establishes consulting office

Anton A. Goldes has established the firm of A. A. Goldes & Associates, Toronto, for the practice of consulting engineering in structural design. Mr. Goldes was formerly chief engineer with the firm of Yolles & Associates, Toronto.

TAC announces tentative program for 1960 ACI convention

Following the May, 1959, meeting of the Technical Activities Committee at Institute headquarters in Detroit, Walter J. McCoy, chairman of TAC, announced a tentative technical program for the 56th annual meeting of ACI to be held in New York City, Mar. 14-17, 1960.

While the program is now at its earliest stage of planning, six technical sessions have been scheduled. The first 1½ days will be devoted to technical committee meetings. The technical program will begin on Tuesday afternoon, March 15, with several sessions running concurrently through Thursday.

The first session will feature proposed ACI standards and committee reports.

Concurrent sessions on March 16 will be devoted to model tests, highways, materials, and design and structural research. On March 17 Committee 201, Durability of Concrete in Service, will present a symposium on restoration of deteriorated concrete. Hubert Woods, Portland Cement Association, Chicago, is committee chairman. Committee 115 will hold its annual research forum on Thursday afternoon.

Although the Technical Activities Committee has set a deadline of July 1 for acceptance of papers for presentation at the 56th annual meeting, since the program is still in the formative stage, papers of pertinent interest can still be studied for inclusion in the final technical program. Members who feel they have a worthwhile contribution to make should write to Institute headquarters immediately, furnishing a synopsis which makes clear the scope of the proposed paper and indicates features that the author thinks justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and final acceptance by Sept. 15, 1959. Final manuscripts of accepted papers will be due Jan. 1, 1960

Prescon opens Dallas office

Prescon Corp., manufacturer of Prescon post-tensioning system, recently opened an office in Dallas with Kenneth Braselton in charge. Prescon now has offices in nine American cities and in Mexico City, Mexico.

Marietta Concrete merges with American-Marietta

Marietta Concrete Corp., Marietta, Ohio, has announced the merger of its business and operations with American-Marietta Co.

In Chicago, Robert E. Pflaumer, president of American-Marietta, said that Marietta Concrete would be operated as part of the parent company's concrete products division. F. Leonard Christy, president of Marietta Concrete, will continue as the chief operating officer; R. Neil Christy will maintain responsibility for engineering and production, and the entire organization will remain intact.

BRAB surveys house performance

The Building Research Advisory Board of the National Academy of Sciences—National Research Council, Washington, D. C., at the request of the Federal Housing Administration, has undertaken a nationwide pilot survey to identify those areas of a house where scientific and technical knowledge may improve future home construction.

All information will be correlated and analyzed to form a more scientific basis for building research programs designed to improve the quality of future house construction.

Round "block houses" built with demountable traveling forms

STEEL FORMS and a rubber-tired traveler—sectionalized small enough that parts will pass through a 6 x 8-ft doorway—proved vital in construction of blastproof concrete control rooms at the Air Force Missile Test Center, Cape Canaveral, Fla. Construction of the four concrete "block houses"—planned to withstand an impulse loading of 1600 psi—marked first use of these inside, outside, and bulkhead surfacing forms.

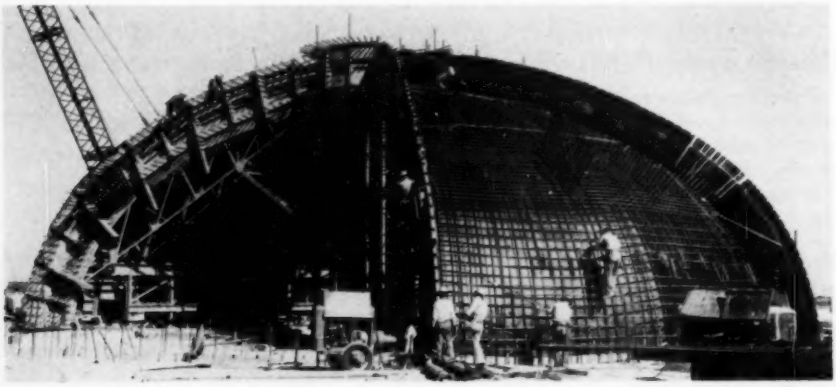
One of the dome structures—26 ft high and 80 ft in diameter—is located 750 ft from the static firing and launch stands, and houses control and recording equipment and personnel necessary for missile testing and launching.

Stability and resistance to overturning led to the selection of a concrete dome as the type of structure best able to withstand ultra-high blast pressures. Through malfunction or intentional destruction of a missile, an explosion could be possible that would produce destruction equivalent to that caused by 50,000 lb of dynamite. The 5 to 6½ ft thick concrete dome is also covered with 5 to 7 ft of sand and a 4-in. shotcrete surface.

A total of 1900 cu yd of 3750-psi concrete was used in the concrete dome, which was placed in three sections. Actually, concrete was placed 6 ft 7 7/16 in. thick at the base and narrowed to 5 ft at the crown. Reinforcement was 1⅛ to 1½ in. diameter bars. Each section covered 120 deg—60 deg on opposite sides of a central compression ring. Each 60-deg arc was approximately 43 ft wide inside and 49 ft wide at outside base.

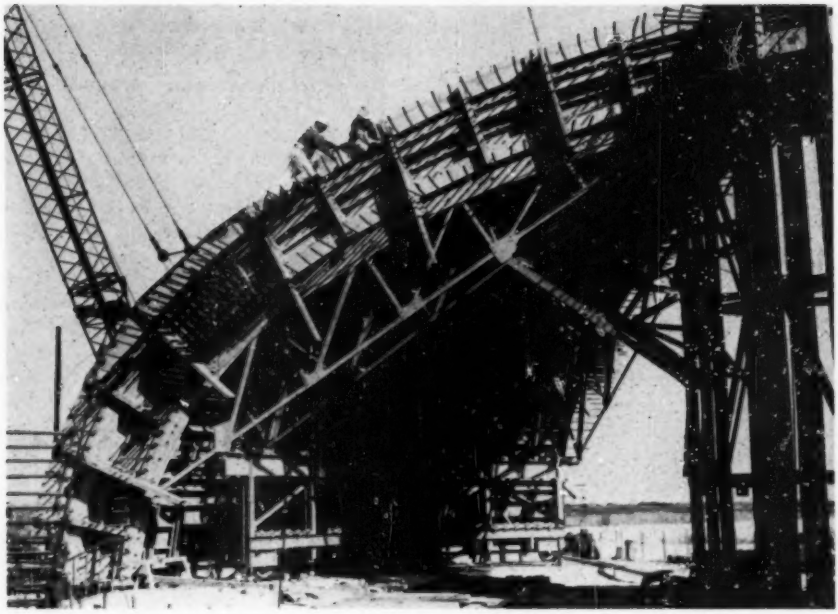
Construction sequence

Concrete was first placed in a 5 ft deep, 12 ft diameter central compression ring. Temporary support for the ring was provided by 12 wide flanged steel beams.



Photos—U. S. Air Force

Steel workers set reinforcing for the 5 to 6½ ft thick concrete dome. The completed dome has a height of 26 ft and a diameter of 80 ft. Outside forms were required for only the bottom two-thirds of the dome. Traveling forms were used for the interior surface



This closeup shows the method devised for moving inside forms. Screw jacks under the travelers lowered the form vertically until rubber tires of the travelers touched the concrete slab. Stripped formwork then moved circumferentially to the next position where screw jacks raised the form into position

For each of the 120-deg placements, the cut pie-shaped forms varied from panels 12 ft wide at inside base to 1 ft at inside crown. All inside forms were braced by steel framework mounted on rubber-tired travelers which moved over a 5 ft thick, 12 ft wide concrete floor slab. The travelers were 6 ft from forms at inside base. Struts were employed at the top of each traveler and anchored to the temporary center pedestal.

Outside surface forms also measured 12 ft wide at the base, and narrowed to 4 ft two-thirds of the distance between base and crown. Outside surface forms had to reach only this far because the upper third of the outside surface approached the concreting conditions of a flat slab. The orange peel sections for the lower two-thirds of the dome's exterior were positioned by two truck-mounted cranes, equipped with 100 ft long booms.

Bulkhead panel sections for each of four 60-deg placements measured 3 x 5 ft. Twelve sections were required for each 46 ft long bulkhead arc, bolted into position at the central compression ring. Form panels measuring 5 x 5 ft, were used to form the duodectagon cap for the dome.

A complete cycle for setting two 60-deg dome segments consumed 6½ days. Once forms for the initial two segments were positioned, concrete was placed in 7½ hr. After 4 days, workers simultaneously stripped inside and outside surface forms and the bulkhead forms. Screw jacks under two 28 ft long travelers lowered inside formwork until rubber tires touched the concrete slab. The travelers' rubber wheels were set at an angle to allow the forms to traverse circumferentially to the next segment. In position, the jacks were raised again to reposition the forms. This operation consumed 12 hr. Cranes were employed to strip and reposition outside surface and bulkhead forms in 4 hr.

After the last of three 120-deg segments were completed, outside forms were removed.

Then, inside surface forms and related framework, the travelers, and falsework for the central compression ring were disassembled and removed in less than 3 days through the 6 x 8-ft entrance for personnel.

From the erection of forms for the first placement, it took 48 days to complete each block house.

Credits

Diversified Builders, Inc., Montebello, Calif., was the contractor; Russell L. Foster was project manager. Blaw-Knox Co. designed and fabricated the steel forms.

Second World Conference on Earthquake Engineering

Papers on earthquake engineering from as many countries as possible are being solicited by the organizing committee for the Second World Conference on Earthquake Engineering to be held July 11-18, 1960, at Tokyo and Kyoto, Japan. The conference is being organized by the Science Council of Japan in cooperation with the Japan Society of Civil Engineers, the Architectural Institute of Japan, and the Seismological Society of Japan.

An invitation is extended to scientists and engineers throughout the world who are interested in the earthquake program to participate in this international assembly.

Closing date for submitting papers is Jan. 15, 1960. All papers are to be in English. Further information is available from the Organizing Committee, Second World Conference on Earthquake Engineering, Science Council of Japan, Ueno Park, Taito-ku, Tokyo, Japan.

Narramore appointed managing director of Perlite Institute

Perlite Institute, the international trade association of perlite producers, officially observed its 10th anniversary this month and named Marve Narramore as its new managing director. He was previously assistant secretary-treasurer of the Valve Manufacturers Association and has had 12 years experience in trade association management and training.

Soiltest wins public relations award

The international public relations program carried out by Soiltest, Inc., Chicago, was announced as a winner of a Certificate of Achievement at the annual convention of the American Public Relations Association. Soiltest executives visited many countries in Europe, the Middle East, the Near East, South and Central America and the Caribbean area. They met with hundreds of engineers, professional people, the press and government officials to discuss some of the mutual problems and advances in the field of civil engineering. Films were used to demonstrate examples of the engineering and scientific activity in the United States.

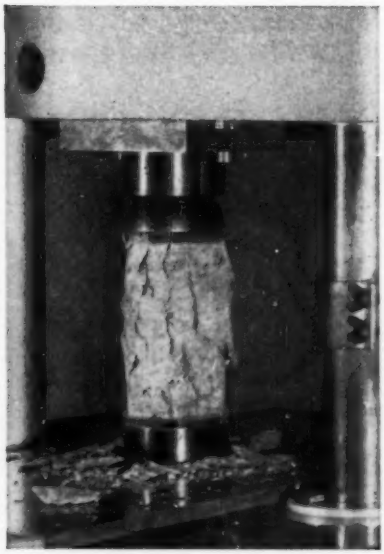
New construction hits all-time high

Total spending for new construction in the first quarter of 1959, a record of $10 billion, showed a 12 percent increase over the same period last year, *Architectural Forum* recently reported.

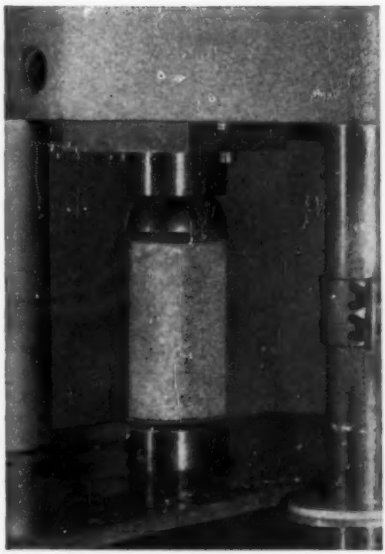
Construction on new dwelling units jumped 32 percent over the first quarter of 1958. However, according to Forum's report, private nonresidential construction, actually trailed last year's first quarter by about 5 percent. The drop in nonresidential activity is attributable to the continuing slump in industrial and office building. Other areas of private nonresidential building that showed minus signs in the first quarter were hospital building and the building of telephone and telegraph facilities.

Running counter to the generally downward trend of nonresidential building, is the construction of stores, restaurants and garages; store building shows a rise of 16 percent for the first quarter of 1959. In the area of public building (up 16 percent in the first quarter to a record of $3.1 billion), only school building dropped slightly from last year's first quarter.

Highway building boomed 23 percent above last year's similar period, and $6 billion total for highway spending in 1959 seems a certainty according to *Forum*. This year as much as 50 percent of total highway spending may come from federal sources, compared with about 20 percent in 1956.



3100 psi Plain Concrete



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Use Johns-Manville liquid Placewel . . . less than 3 fluid oz./sk. of cement

TEST DEMONSTRATES why Johns-Manville Placewel® has been used in over 5,000,000 cu. yds. of concrete. With this liquid admixture, it is possible to use a minimum paste content . . . and get maximum concrete quality.

With Placewel, you use the minimum amount of paste needed to lubricate the mix and bind the aggregate particles together in the hardened state.

By eliminating excess paste you reduce costs, minimize drying shrinkage and make concrete less susceptible to attack by aggressive solutions and weathering.

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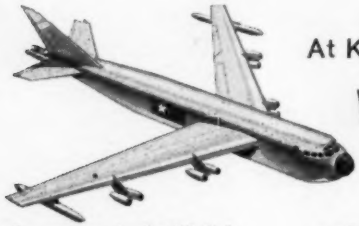

At Kinross Air Force Base . . .

World's heaviest

THE "JET AGE" IS HERE. And with it is the problem of re-building airports to provide the longer, stronger, heavier runways these big jets need. That's why airport builders and designers are planning thicker, stronger airfield pavements with extra-heavy reinforcement . . . pavements that will match or exceed present jet requirements. And that's why the heaviest welded wire fabric ever produced was installed at the vital Kinross Air Force Base in Michigan.

Operational jets in commercial service now impose gear loadings in the neighborhood of 130,000 lbs., with gross weights of 295,000 lbs. Future models may go even higher. For that reason, only reinforced concrete provides the added strength to meet the unusually heavy and severe requirements of the "jet age." Specify USS American Welded Wire Fabric for reinforcing all runways, high speed turnoffs, taxiways, and service pads . . . get these important advantages:

- USS American Welded Wire Fabric distributes heavy wheel loadings over a larger area to prevent harmful overloads at any one point. Impact, shock, suddenly applied or released loads such as occur during landings and take-offs are more uniformly "absorbed" by each slab. Reinforced concrete slabs are 30% stronger than unreinforced slabs of equal thickness.
- USS American Welded Wire Fabric prevents harmful cracking from developing because of its uniformly spaced high-tensile-strength welded steel wire construction. Its action "knits" a slab together.
- USS American Welded Wire Fabric unifies the action of any slab when it expands and contracts due to temperature change.
- Longer, wider slabs can be laid when USS American Welded Wire Fabric is used. This means fewer joints, fewer load transfer assemblies. The reduction in the number of joints not only means economy, but also a smoother, safer landing surface.

When existing airfield pavements must be strengthened, USS American Welded Wire Fabric reinforced concrete overlays will produce unbroken grade lines and provide additional strength. Thus, maximum economy is achieved.

Specify USS American Welded Wire Fabric. It's available in a wide variety of styles, lengths, and widths . . . in wire sizes from ½" diameter to 16 ga. and in longitudinal or transverse wire intervals of 2" to 16". The engineers at American Steel & Wire will be glad to tell you more about the application possibilities of fabric and how it can serve your needs. Get in touch with American Steel & Wire, Dept. 9134, 614 Superior Avenue, N.W., Cleveland 13, Ohio.

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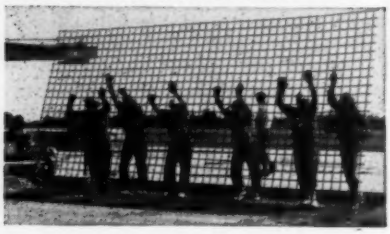

Contractor: Loselle Construction, Inc., Wyandotte, Michigan
Paving Material Supplier: Scioto Supply Company, Lansing, Michigan
Design and Supervision: Corps of Engineers, U. S. Army

At Kinross Air Force Base, workmen are installing a section of USS American Welded Wire Fabric—Style 55-7/07/0 (7/0 Gauge is 0.490" Diameter Wires). USS American Welded Wire Fabric is entirely fabricated by electrically welding all intersections of the high strength steel wires. This insures positive placement of the steel in the slab and eliminates costly field tying and extra handling.

Welded Wire Fabric helps the big jets land safely!

Under the direction of the U. S. Corps of Engineers, another new United States Air Force Base is made ready to join our air defense network. It is Kinross, located on the upper Peninsula of Michigan. This base was expanded to handle bigger, heavier modern jet aircraft. USS American Welded Wire Fabric was used in the rigid overlays on the strengthened and lengthened runways.

USS American Welded Wire Fabric

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DUMBBELL TYPE

Servicised Polyvinyl Plastic Waterstop—made from high grade polyvinylchlorides and plasticizers—is an extremely stable material that is easy to handle, install and splice in the field. It has the flexibility, durability and superior aging qualities necessary in critical waterstop applications.

Servicised Polyvinyl Plastic Waterstop is available in 4″, 5″, 6″ and 9″ Flat Dumbbell, and 9″ Centerbulb types. 4″ and 5″ size is supplied in lengths of approximately 100 lineal feet; 6″ and 9″ sizes in lengths of approximately 50 lineal feet.

Special Polyvinyl Plastic Waterstop circular has complete details and physical data. Write for it today.

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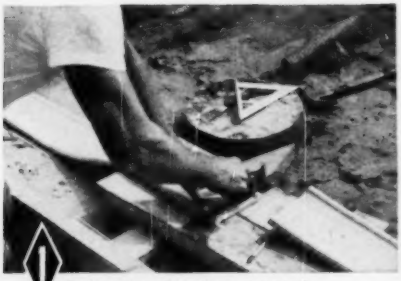

1 Ends to be spliced are squared.

2 Place ends on heated plate and hold firmly until material begins to melt.

3 Join ends immediately, maintain slight pressure until material cools.

4 Completed splice becomes integral part of waterstop.

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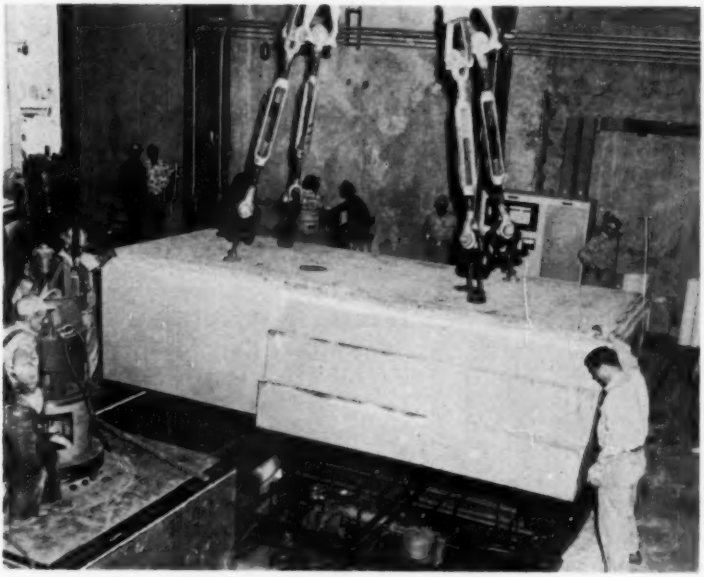


Super-Block for Radiation Shielding

cast-in-place magnetite concrete also used in reactor housing

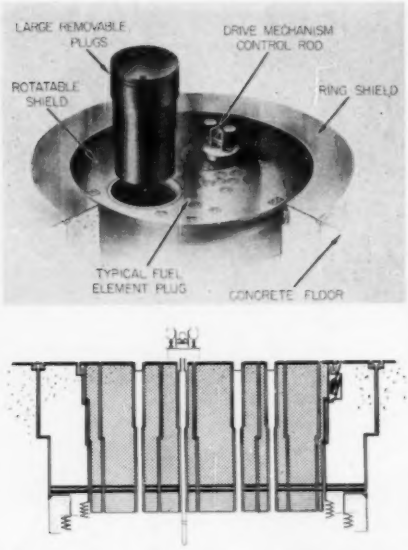
MAGNETITE ORE CONCRETE blocks 5½ ft thick were installed over primary sodium galleries of the Sodium Reactor Experiment near Canoga Park, Calif. The blocks form a gastight seal with the edge of the adjacent floor and with each other. Sides are stepped to prevent radiation streaming.

High-density concrete made with magnetite aggregate also forms the biological shield for the plant's nuclear reactor. The reactor core tank is encased in a 5½-in. thermal shield of cast steel and a ¼-in. steel outer tank which can contain sodium in the event of a leak in the core tank. This outer tank is in turn surrounded by 12 in. of thermal block insulation, a ¼ in. thick carbon steel liner, and 3 ft of high-density concrete. The liner served as a form for the concrete, and has 28 1-in. diameter cooling pipes attached to remove the small amount of heat generated in the concrete, plus the heat conducted across the thermal insulation barrier. The base of the liner rests on a pad of concrete 29 ft below the floor level.

The reactor's top shield is of high-density concrete, 6 ft thick. It provides access to elements in the core through stepped casings for 81 small plugs and several larger ones. Detail drawing shows stepped sides of these casings, and stepped side of the full top shield which weighs about 75 tons.

Thermal radiation to the underside of the top shield is reduced by a series of horizontal, thin stainless steel plates suspended from the shield. Then there are successive layers of steel, lead, and more steel—a 1-in. layer which is the final thermal shield. Above this the magnetite ore concrete extends to the top of the shield.

Concrete tilt-up wall panels were used in the steel framed building which houses the



Detail of the top shield which stops radiation above the Sodium Reactor Experiment, as well as a means of access to elements in the core. Shield is filled with magnetite ore concrete

reactor experiment; structure measures some 50 x 100 ft, with 45 ft high bay area overhead.

The Sodium Reactor Experiment was designed and constructed by Atomics International, a division of North American Aviation, Inc., for the Atomic Energy Commission as part of a program to develop the basic technology for competitive nuclear power plants. Southern California Edison Co. has installed a 7500-kilowatt steam generating plant which uses the reactor heat to produce electricity.

Pierson moves to Marquette Cement

C. U. Pierson, Jr., is now technical director for Marquette Cement Manufacturing Co., Chicago. He was formerly associated with Southern Cement Co., Division of American-Marietta, Birmingham, Ala. Mr. Pierson is a member of ACI Committee 331, Structures of Concrete Masonry Units.

AGC headquarters building wins civic award for design

Civic awards for design and appearance of the national headquarters building of the Associated General Contractors of America in beautifying Washington, D. C., were made recently to the architect, Chatelain, Gauger and Nolan, of Washington; the general contractor, Joseph F. Nebel Co., Washington; and AGC President James W. Cawdrey, Seattle, on behalf of the association.

Laclede Steel presents two 1959 student engineering awards

George C. Lindh of St. Louis University and Arlys T. Peterson of Washington University recently received the 1959 Laclede Steel Co. award as the outstanding senior civil engineer in their respective universities. This award is based on scholastic accomplishment, extra curricular activities, and specific professional interest in student organizations and professional societies.

Hale Sutherland

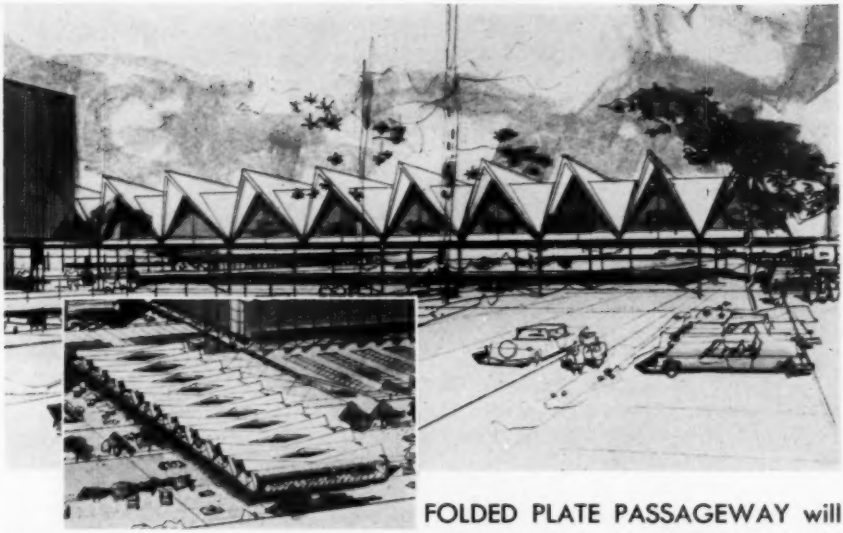
Hale Sutherland, professor emeritus of civil engineering at Lehigh University, died the latter part of May, at the age of 74.

He retired from the Lehigh faculty in 1953 after a teaching career which spanned 40 years; 23 at Lehigh and 17 at the Massachusetts Institute of Technology.

As a member of several national committees of the American Society for Engineering Education and the American Society of Civil Engineers, he had been influential in establishing humanistic social courses in the engineering curricula of colleges and universities of the nation.

Professor Sutherland received his bachelor's degree from Harvard College in 1906 and his civil engineering degree from MIT in 1919. He was the coauthor of two text books: *Structural Theory and Design* with H. L. Bowman and *Reinforced Concrete Design* with W. W. Clifford (later editions coauthored with Raymond C. Reese).

He had been a member of ACI for over 40 years and was affiliated with the National Association of Professional Engineers.



FOLDED PLATE PASSAGEWAY will span mall area and street at the Electronics Center, Santa Monica, Calif. The thin shell concrete structure, which will connect two segments of the center, is part of a $38 million expansion project developed by John M. Stahl, industrial developer. Tilt-up concrete exterior walls are to be used on the other four structures in the expansion project. Richard A. Dorman is the architect.

Ekelund joins Cincinnati Milling Machine staff

Bror S. Ekelund, formerly assistant professor of chemistry, South Dakota State College, Brookings, has been named senior research supervisor of ceramics and abrasives in the chemistry research department of Cincinnati Milling Machine Co.

Rayner elected president of Gulf consulting group

Alfred G. Rayner, consulting engineer of Baton Rouge, La., has been named president of Gulf Institute of Consulting Engineers.

Other officers include: T. Edward Ernst, New Orleans, first vice-president; B. M. Dornblatt, New Orleans, second vice-president; and John P. Jones, Baton Rouge, secretary-treasurer.

Organized in 1950, the Gulf Institute of Consulting Engineers is a professional organization, whose members are registered private consultants in the fields of civil, mechanical, electrical, and chemical engineering. Membership covers the four-state area of Arkansas, Louisiana, Mississippi, and Alabama.

Officers installed at 39th annual SAME meeting

The following newly elected officers of the Society of American Military Engineers were installed at the 39th annual meeting of the group, May 18, 1959: President, Vice-Adm. W. Orme Hiltabidle, CEC, U. S. Navy, Retired, vice-president, Charles H. Thompkins Co., Washington, D. C.; first vice-president, Maj. Gen. Augustus M. Minton, director of civil engineering, U. S. Air Force, Washington, D. C.; northeastern regional vice-president, John A. Reilly, vice-president and director, Arundel Corp., Garden City, New York; and midwest regional vice-president,

Danial A. Sullivan, consulting engineer, Armour Research Foundation of Illinois Institute of Technology, Chicago, Ill.

ASEE holds 67th annual meeting

More than 2000 members attended the 1959 annual meeting of the American Society for Engineering Education in Pittsburgh, June 15-19. The University of Pittsburgh and Carnegie Institute of Technology were co-hosts for the meeting.

Principal addresses at the convention, which included over 125 conferences, meetings, and other sessions, were centered on increasing the effectiveness of engineering education to meet the needs of graduates in professional careers which will extend over the next 45 years.

Benjamin R. Teare, Jr., Dean of the College of Engineering and Science at Carnegie Institute of Technology, was installed as president for the year 1959-60.

LOOKING AHEAD

Aug. 10-12, 1959—Conference of Lightweight Concrete Block Manufacturers, National Cinder Concrete Products Association, Chalfonte-Haddon Hall, Atlantic City, N. J.

Sept. 21-25, 1959—International Council for Building Research, Studies, and Documentation (CIB) Congress, Rotterdam, The Netherlands

Oct. 29-30, 1959—42nd Annual Meeting, National Slag Association, Boca Raton Hotel and Club, Boca Raton, Fla.

Nov. 2-6, 1959—5th Annual Convention, Prestressed Concrete Institute, Deauville Hotel, Miami Beach, Fla.

Nov. 2-5, 1959—Regional Meeting, American Concrete Institute, Hotel Continental Hilton, Mexico City, Mexico

National Gypsum acquires Huron Portland Cement

National Gypsum Co. of Buffalo recently exchanged more than one million shares of stock, valued at some $67 million to acquire Huron Portland Cement Co. with headquarters in Detroit.

National Gypsum's new cement making subsidiary will continue under the direction of president Paul H. Townsend, who has been Huron's chief executive for many years. The company will have its own sales force, production units, accounting and research facilities, and will continue operating under the name of Huron Portland Cement Co.

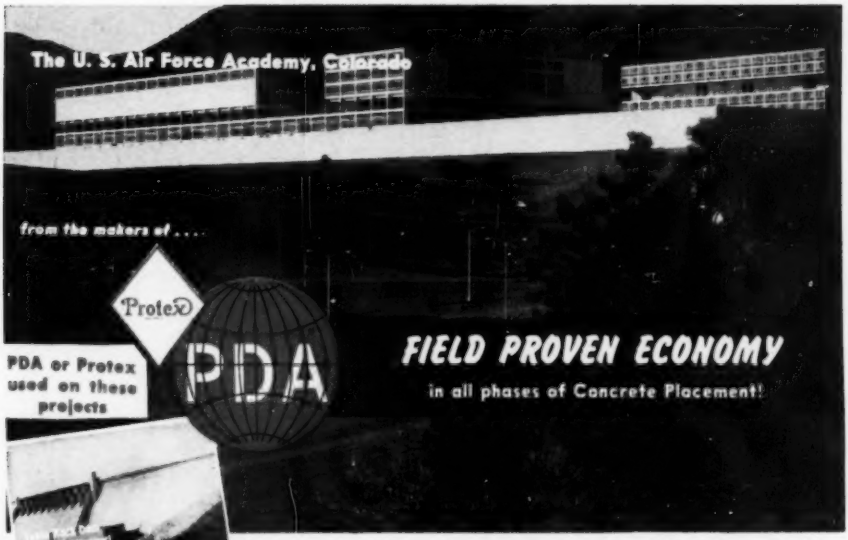
DeShazo appointed Alpha southern sales manager

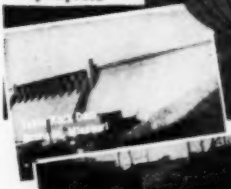
Manon DeShazo, Jr., has been appointed southern sales manager for Alpha Portland Cement Co., Easton, Pa. He has been a salesman in the firm's Birmingham sales district. Mr. DeShazo succeeds H. E. Brown, who was recently elected vice-president in charge of Alpha's midwestern sales division.

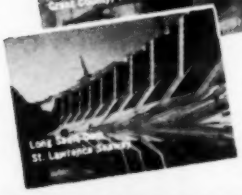
The company also announced the addition of five new salesmen to its field sales force. Richard R. Reese, Fred Mullineaux, and Robert Holden join Alpha's eastern district office; John C. Woodruff joins the New York district office; and Havard E. Griffith has been appointed salesman in the central Pennsylvania area.

Examinations open for patent examiner and patent adviser

The U. S. Civil Service Commission has announced two open examinations for: (1) patent examiner in the field of engineering and applied sciences, (2) patent adviser in the Patent Organization of the Navy Department in Washington. Appropriate college study in engineering or physics will meet the qualifications for both these positions at the entrance level. Professional experience in a pertinent field of work is required for the higher paying positions. Forms may be obtained from Civil Service regional offices or from the United States Civil Service Commission, Washington, D. C.


The U. S. Air Force Academy, Colorado
from the makers of . . .
Protex
PDA
PDA or Protex used on these projects
FIELD PROVEN ECONOMY
in all phases of Concrete Placement!





Among the many noteworthy and well-known projects around the world that are using PDA stands the name of the Air Force Academy in Colorado. Joining with this magnificent installation are dams, missile sites, highways, airfields, buildings . . . any construction where the efficient and economical placement of concrete is important!

PROTEX Dispersing Agent . . .

PDA increases strength at any age through water reduction and cement dispersion, improving all the desirable characteristics of concrete, giving a "live" concrete with protection against "hot" weather" slump loss and segregation — allows the placing of controlled durable air entrained concrete without plastic shrinkage cracks. PDA is a selective initial retarder— retarding only the initial set of concrete for extended vibration and finishing time (not delaying form stripping time) YET it gives no retardation in winter concreting — thus year around benefits are obtained with "all season" PDA.

Proven, Dependable, Adaptable

Proven in the field . . . PDA insures the successful and economical placement of better quality concrete!

Dependable — YES! Proof positive from Government and private projects. Also backed by the world-known and world-respected PROTEX name!

Adaptable to any need . . . PDA improves any Pre-stressed, Slip-form, Light-weight, Tunnel-lining, Tilt-up or Lift-slab concrete project!

PDA packaged in a durable bag comes to you in convenient powder form . . easily mixed and dispensed . . complete dispensing installations available upon request

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FOR BETTER AND MORE ECONOMICAL PLACEMENT OF CONCRETE, SPECIFY AND USE PDA . . . FROM THE MAKERS OF PROTEX!

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Attention of:_____

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*T.M. Reg.

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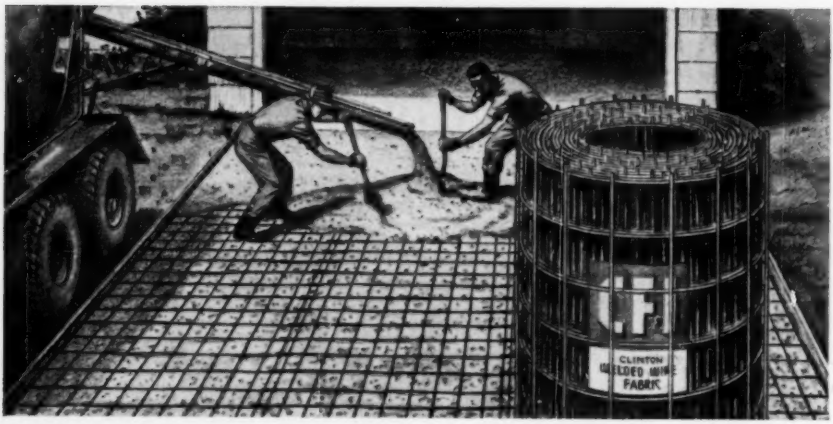

Concrete is good...

Reinforced **concrete is better**

The CF&I Image—symbol of many modern steel products for the construction industry—provides CF&I-Clinton Welded Wire Fabric for crack-resistant concrete work. A steel skeleton adds tensile strength to the concrete, allowing it to cushion the shock imposed by heavy loads. As it absorbs the stress, the steel fabric distributes it evenly in all directions. Thus, the impact is never concentrated in one area.

Steel fabric reduces the chances of cracking during setting. As it holds the concrete tightly together, it helps eliminate heaving due to extreme temperature changes. Should a tiny surface fissure develop, the steel fabric acts like a vise, preventing dirt or moisture from expanding the crack.

For complete information about economical, easy-to-install CF&I-Clinton Welded Wire Fabric, contact the nearest CF&I office.

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THE COLORADO FUEL AND IRON CORPORATION

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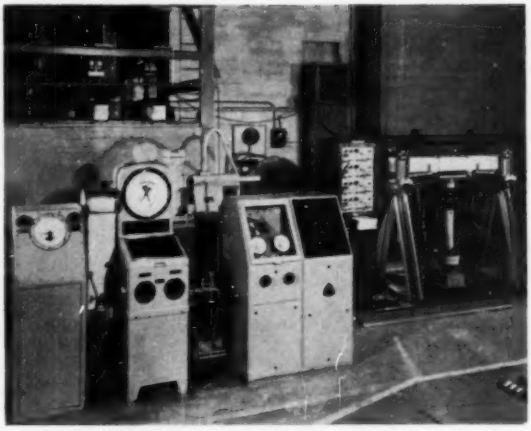
CF&I OFFICE IN CANADA: Montreal • **CANADIAN REPRESENTATIVES AT:** Calgary • Edmonton • Vancouver • Winnipeg

CUMULATIVE FATIGUE DAMAGE IN CONCRETE is subject of a 3-year study being conducted by the Department of Theoretical and Applied Mechanics, College of Engineering, at the University of Illinois under the direction of Prof. Clyde E. Kesler. The National Science Foundation is sponsoring the project.

Believed to be the first to be conducted in the United States, this investigation is to study plain concrete specimens subjected to flexural stresses of varying magnitude and sequence.



Results of these fatigue tests are expected to yield valuable new information about the general behavior of concrete subjected to loads of varying magnitude followed by rest periods or high static overloads. For example, conditions are simulated like those to which a concrete highway is subjected during its lifetime.

Photograph shows the new Losenhausen-Riehle fatigue testing equipment to be used in the series of tests. Setup includes control consoles (at left), programmer (center), pulsator (rear), and specimen over loading cylinder in test stand (right). Results are being obtained from 6 x 6-in. concrete beams loaded at the third points on a 50-in. span.

Mason contractors in Dallas to host statewide meeting

Masonry contractors from throughout Texas will convene in Dallas on September 4 and 5 for the organizational meeting of the Mason Contractors Association of Texas.

W. B. Slayden, Dallas masonry contractor, and state vice-president for the Mason Contractors Association of America, will serve as convention chairman. The Dallas chapter, headed by Dee Brown of Dee Brown Masonry, Inc., will be host chapter for the first meeting.

NCSA joins CAP-AGC joint committee

The concrete and aggregate producers side of the CAP-Associated General Contractors Joint Cooperative Committee has grown to three members with the addition of the National Crushed Stone Association. The other two are the National Sand and Gravel Association and the National Ready Mixed Concrete Association.

CAP-AGC held its first exploratory meeting in Miami Beach last January. The next meeting is scheduled for later this year. R. W. McKinney, AGC, Nacogdoches, Tex., is chairman of the committee and James M. Sprouse, manager of the AGC Highway Division, is secretary.

Errata

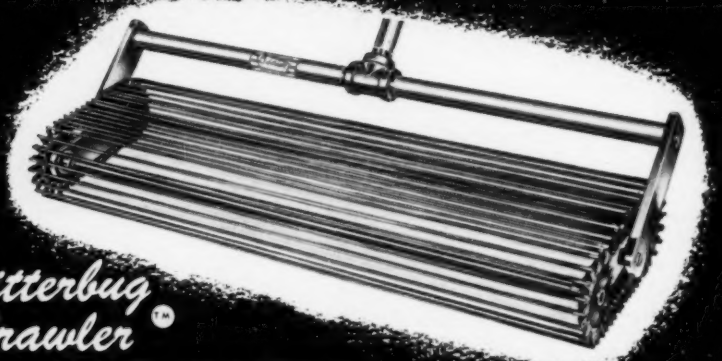
The following corrections should be made in "Effect of Powdered Minerals and Fine Aggregate on the Drying Shrinkage of Portland Cement Paste," by Kenneth M. Alexander and John Wardlaw, which appeared in the June, 1959, JOURNAL.

p. 1306—for the reference in the last line read 15 instead of 13.

p. 1315—the formula under the figure should be corrected to read

$$\log \Delta S'_{O\text{-}R} = \log \Delta x_{O\text{-}R} + a'R + b'$$

New Concept for CONCRETE!



Jitterbug Crawler™ CONCRETE TRACTOR

NO OTHER DEVICE EVER DEVELOPED FOR THE PREPARATION OF POURED CONCRETE SLABS COULD EVER MAKE

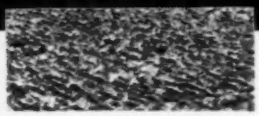
These Proven Claims . . .

- **TRACTORING FLOATS MEDIUM AGGREGATE** Speeds preparation time where stiff slumps are poured, for top quality concrete.
- **TRACTORING MAKES "PASTE" SURFACE . . FAST** Longer contact with mix keeps coarse aggregate down until "fines" rise around and above.
- **TRACTORING ENDS WADING IN THE MIX** Because you 'tamp" a large area with an easy push-pull motion, jitterbugging goes many times faster.
- **TRACTORING "LIBERATES" ONE MAN** Since it's so easy to use, you can use your jitterbug man for many other jobs besides tamping.
- **TRACTORING MAKES NEW TEXTURES POSSIBLE** New textures and new traction surfaces available without costly preparation.

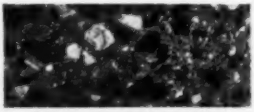
CRAWLING JITTERBUG™

CONCRETE TRACTOR MEANS BETTER CONCRETE . . . IN LESS TIME . . . AT LOWER COST!

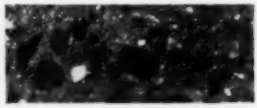
Precision engineered and fully job-proven, the Crawling Jitterbug Concrete Tractor consists of 46 parallel steel rods, each 35" across, operated by gear-driven sprocket chain. Easily operated by one unskilled man, without fluctuation in performance caused by fatigue! Completely assembled, with 12-foot sectional handle, ready to use on the job. Complete, $115.00, FOB Factory in Kansas City. Or write for FREE descriptive booklet on this sensational new machine!



Close-up photo of Tractored surface shows how concrete Tractor's parallel bars press straight down into mix, allowing medium and fine particles to rise around them. No other method leaves surface prepared as shown in this photo!



Sawed cross-section of ordinary slab which was not Tractored shows how the coarse aggregate lies too close to finished surface — ready to peel and spall. Ordinary preparation methods do not let fine and medium particles rise around and above coarse aggregate. Over-working of mix may depress coarse material too far and weaken slab.



Sawed cross-section of Tractored slab shows even distribution of coarse aggregate. Coarse material is held just below surface, to permit rise of perfect paste. But distribution is even, without settling of coarse material to bottom of mix, which would alter monolithic structure of finished concrete.

Goldblatt TOOL COMPANY

FREE Booklet describes new method completely—provides facts and specs on this great machine! Write for your copy, today!

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Honor Roll

January 1—June 30, 1959

Robert P. Witt is again in first place with 37½ credits. Samuel Hobbs is second with 34 credits.

Be an ACI booster. The new member you propose may bring important information to ACI, and ACI will reciprocate generously from its reservoir of technical data.

Robert P. Witt	37½
Samuel Hobbs	34
Faraj Tajirian	13
George B. Southworth	11
Charles C. Luther	10
David A. Sauer	10
James Chinn	9
Chas. W. Cole, Jr.	9
Kenneth M. Huber	9
A. T. Klassen	9
Francesco Sardella	9
H. C. Pfannkuche	7
Henry Aaron	6
Robert F. Adams	6
Jose Maria Bravo	6
John L. Burdick	6
S. D. Burks	6
Mauro N. Guzman	6
John E. Heer, Jr.	6
Horace G. Hill, III	6
Aleck E. Hiscox	6
Elias Kardaras	6
George J. Kerekes	6
George E. Large	6
Leo M. Legatski	6
W. S. Cottingham	5
Gregorio Hernandez	5
Jack C. McCoe	5
Gene M. Nordby	5
C. P. Siess	5
Ferruh Taskin	5
Michael Alexander	4
W. H. Armstrong	4
Ira M. Beattie	4
George P. Duecy	4
Phil M. Ferguson	4
Eberhart Gunther	4
Martin J. Gutzwiller	4
B. E. Kester	4
S. J. Moore	4
Wendell H. Nedderman	4
Constantin Polidoroff	4
Jerome M. Raphael	4
Joseph J. Shideler	4
M. F. A. Siddiqui	4
Howard Simpson	4
James E. Stanners	4
Glenn C. Thomas	4
Lewis H. Tuthill	4
Victor Achim	3
Jay B. Ames	3
Tamnoon Ansusinha	3
John D. Antrim	3
Edward E. Baker	3
George Batievsky	3
Dutton Biggs	3
J. G. Bodhe	3
D. E. Bowen	3
Dan E. Branson	3
John E. Breen	3
Ralph B. Brenan, Jr.	3
C. F. Brown, Jr.	3
Theodor H. Busck	3
G. W. Butcher	3
Guillermo Castellanos G.	3
T. J. Cavanagh	3
Solomon Chornik S.	3
Marvin B. Cohen	3
Ralph G. Crimms, Jr.	3
Frederick W. Drury, Jr.	3
Thomas A. Duwelius	3
Gene E. Ellis	3
B. A. Eskenazi	3
R. H. Evans	3
James Lee Ford	3
Ray J. Foss	3
Russell H. Fuller	3
Junius R. Gardner	3
Ben C. Gerwick, Jr.	3
Raleigh DeVisme Gipps	3
Myron L. Goral	3
Louis A. Gottheil	3
Robert B. Harris	3
Eric J. Hayford	3
Howard H. Hays	3
F. J. Hosgood	3
Samuel I. Iwata	3
J. E. Jellick	3
J. E. Jones	3
Truman R. Jones, Jr.	3
William W. Karl	3
Joseph Karni	3
Allan A. Kay	3
Joe W. Kelly	3
Frank Kerekes	3
Narbey Khachaturian	3
Frank R. Killinger	3
Solomon Kirschen	3
William C. Krell	3
Robert R. Kuske	3
Edward Laing	3
Ronald Lazar	3
Elmer C. Lee	3
Janis Liepins	3
Henry Lipkind	3
Russ A. Loveland	3
Douglas McHenry	3
Keith E. McKee	3
Ernst Maag	3
M. F. Macnaughton	3
Frank B. May	3
Eduardo Mercado Flores	3
William J. Moore	3
Newlin D. Morgan, Jr.	3
Leslie E. Moss	3
W. E. Moulton	3
Donald H. Olson	3
Rafael Enrique Pacheco	3
Diego Parra Pardi	3
Orris O. Pfutzenreuter	3
Jack L. Randall	3

E. B. Rayburn, Jr. 3
Thomas J. Reading 3
Raymond C. Reese 3
Sigmund Roos 3
Antonio I. Rosquete 3
Donald A. Sawyer 3
Paul Serfass 3
L. Shector 3
Jehangir C. Shroff 3
Floyd O. Slate 3
Stuart H. Snyder 3
Ralph W. Spencer 3
Leroy A. Staples 3
W. N. Steinmann 3
Willard Stevenson 3
Anthony A. Styner 3
T. Tassios 3
F. D. Taylor 3
Pastor B. Tenchavez 3
J. Antonio Thomen 3
John P. Thompson 3
Sophus Thompson 3
Sylvester J. Turley 3
Ellis S. Vieser 3
Bernardo Villegas 3
L. T. Willoughly 3

Roberto Zepeda Aldana 3
Stanley G. Zynda 3
Pedro M. Bassim 2
Walter E. Blessey 2
Boris Bresler 2
Paul E. Chen 2
P. Q. Chin 2
Miles N. Clair 2
Haraldo Duarte Villela 2
Ambrosio R. Flores 2
D. A. Guntin 2
Oscar Latorre M. 2
Joachim F. Leppmann 2
Leo Liberthson 2
Carl F. Long 2
Ernest C. Marmet 2
Jorge Rodriguez Lebron 2
Emilio Rosenblueth 2
Sabri Sami 2
Nicanor B. Santos 2
Javier Santolalla Silva 2
K. Sitaraman 2
A. M. Tovar 2
Jose R. Perez Valentin 2
C. K. Wang 2
J. H. Weaver 2

New Members

The Board of Direction approved applications in the following catagories: 62 Individual, 2 Corporation, 7 Junior, and 17 Student, making a total of 88 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership on June 1, 1959, was 9910.

Individual

ACKER, BRICE P., El Paso, Texas (Proj. Mgr., C. H. Leavell & Co.)
ANNETT, S. R., Montreal, Que., Canada (Mohawk Chemical Ltd.)
ANTONICH, PETER A., Detroit, Mich. (Engr., Tech. Services, Huron Portland Cement Co.)
ARMSTRONG, E. B., Brownfield, Texas (Mgr., South Plains Ready Mix, Inc.)
BAER, OLIVER A., Berkeley, Calif. (Struct. Engr., Dalton & Dalton)
BERGMEIER, KENNETH E., Topeka, Kansas (Assoc. Engr., Servis, Van Doren & Hazard)
BOLOHAN, JEREMIE, Montreal, Canada (Struct. Designer, Eskenazi, Loebenberg, Slone, Bernstein, Affiliated Cons. Engrs.)
BOOTH, W. J., N. Burnaby, B. C., Canada (Supv. of Concrete Constr., B. C. Engrg. Co.)
BOYT, EUGENE P., Dutch John, Utah (Chf. Mech. Branch, Flaming Gorge Dam, USBR)
BRAUN, J. F., Montreal, Que., Canada (Asst. Dir. of Engrg. & Constr., Aluminum Co. of Canada, Ltd.)
CANFIELD, LEWIS V., Gridley, Calif. (Sec.-Mgr., Reclamation Dist. No. 833)
CARCATERRA, THOMAS, Hyattsville, Md. (Sr. Struct. Engr., Mills, Petticord & Mills)
CHARLES, JULES T., Tulsa, Okla. (Proj. Engr., Tulsa Rig Reel & Mfg. Co.)
CLARKE, GEOFFREY HERBERT, Brisbane, Queensland, Australia (Partner, R. J. McWilliam & Partners)
CRESPI, DR. FILIBERTO, Milano, Italy (Mgr., Admixture Co.)
DALY, RALPH F., Hayward, Calif. (Struct. Engr., City of Hayward)
DOTY, RALPH E., Muskogee, Okla. (Proj. Engr., Corps of Engrs., Tulsa Dist.)
DRAPER, JAMES L., Oscoda, Mich. (Plant Supt., Draper Redi-Mix, Inc.)
GEERTZ, CLIFFORD J., San Francisco, Calif. (City Engr., Chge. of Bur. of Engr., City & County of San Francisco)
GERBER, LORIS L., Englewood, Colo. (Vice-Pres., Rock Mountain Div. Mgr., Prescon Corp.)
GERLACH, S. C., Tulsa, Okla. (Sales Engr., McMichael Concrete Co.)
GIDDINGS, RAY C., Honolulu, Hawaii (Matls. Engr., Honolulu Constr. & Drnying Co., Ltd.)
GORSHA, N. J., Alexandria, La. (Office Mgr., Supv. Field Insp. & Testing of Concrete, Ralph S. Holt Testing Lab.)
GRANT, CALVIN T., Norman, Okla. (Field Engr., PCA)
GRENNAN, PHILIP M., New York, N. Y. (Assoc., Alfred Easton Poor, Archs.)
HAASE, JAMES, Montreal, Que., Canada (Div. Engr., Constr. Engrg. Dept., The Foundation Co. of Canada Ltd.)
HALWAX, FRANK K., Chicago, Ill. (Assoc. Engr., Struct. Analysis Section, Armour Research Foundation)
HANNAN, A. K. M. ABDUL, Dacca, East Pakistan (Exec. Engr., Gov't of East Pakistan)
HARTLEY, WILLIAM SKELTON, Waltham, Mass. (Engr., Struct. Design of Bldgs., Maurice A. Reidy)
HEIM, ORVILLE E., Largo, Florida (Engr., Owner of Concrete Brick Plant)
HEMME, JOHN M., Jr., Sacramento, Calif. (Asst. C. E., State of Calif., Dept. of Water Resources)
HENNY, G. E. J., Johannesburg, South Africa (Sr. Engr., Keeve Steyn, Cons. Engr.)
KIMBALL, WILLIAM T., Ann Arbor, Mich. (Job Supt., Owens-Ames-Kimball Co.)
KOFMAN, ABSALOM, Haifa, Israel (M. Sc. Research Engr., Israel Inst. of Tech.)
KRAVETZ, GLEBE ALEXANDER, Forest Hills, N. Y. (Soils & Foundations Engr., Harza Engrg. Co.)
LAO, JUAN EDUARDO TORRES, Ponce, P. R. (C. E.)
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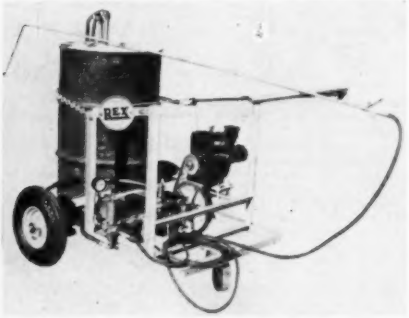
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Portable spray curing machines

Two Rex portable spray curing machines for use in limited or off-slab areas are being manufactured by the Construction Machinery section of Chain Belt Co.



One unit is a hand-carried model with steel base plate and lifting handles; the other unit is a rubber-tired model with a mounting platform for 55-gal. drum.

Both models are equipped with single cylinder, 4-cycle air-cooled gasoline engine; positive displacement pump with automatic by-pass that returns unused materials to drum; stainless steel spray nozzle; 10-ft wand for long reaches; and 25-ft connecting hose that permits operator to work more extensive areas between machines.—Chain Belt Co., Milwaukee 1, Wis.

Combination masonry drill-anchor

Rawl "Saber-Tooth" combination drill-anchor drills its own hole, then becomes a masonry anchor with holding power up to 17,860 lb to fasten fixtures to concrete according to the Rawlplug Co. An expander plug placed in drill end when anchor is driven home expands at bottom of hole. The drill-anchors may be used with power-operated or hand hammers.—The Rawlplug Co., Inc., 248 Petersville Road, New Rochelle, N. Y.

Dry bulk transport

Cement Transport Model BMC-T, self-unloading dry bulk materials transport, highlighted a series of 14 new model trailers and truck bodies introduced by Dorsey Trailers.

The BMC-T has three 20-in. fill hatches with watertight covers. Twin screw discharge system unloads at the rate of 5 bbl per min. Unit is hydraulically powered through tandem pump operated from the tractor PTO. With a 25-hp air-cooled engine, offered as optional equipment, it can operate with any tractor. Available in wide range of capacities and lengths.—Dorsey Trailers, Elba, Ala.

Portable gas space heater

Muller Machinery has announced a portable LP gas forced heat space heater, rated at 200,000 Btu. Unit includes burner control that quickly turns off fuel if power should fail or flow of gas is interrupted. Fan requires 110-v a-c outlet. Operator has complete control of the desired heat output ranging from 0 to 200,000 Btu. Bulletin A39 gives full details.—Muller Machinery Co., Inc., Metuchen, N. J.

Convertible masonry saw

Felker's Touch-A-Matic masonry saw, designed for using 20 in. Di-Lock diamond blade, can also be used with blades as small as 14 in. diameter. Manufacturer states that this saw rips a 16 in. concrete block from one corner diagonally to the other in a single pass.

The Touch-A-Matic has all front end controls, permitting head adjustments and other changes from the operator's position. The saw can be used either wet or dry since it is equipped with a recirculating pump.

Lightweight motors of 1½, 2, or 3 hp may be selected which develop as much as 8 hp on peak loads, intermittent operation. The rack allows any height setting to be made in steps of ¼ in. Units can be used on either 110 and 220 volt ac. Literature and full data available.—Felker Manufacturing Co., Torrance, Calif.

Plastic insert forms contraction joint

Edoco Technical Products, Inc., has developed a rubberlike plastic for insertion in unhardened concrete which, after the concrete hardens forms a weakened plane contraction joint, seals it, and ties slabs together. According to the manufacturer the installation is simple and the material buried in concrete does not deteriorate through weathering and is stable to freezing temperatures up to − 55 F. The company has made experimental installations on several highways.—Edoco Technical Products, Inc., Long Beach, Calif.

One-man vibrators

Champion Manufacturing Co. has entered the concrete vibrator field with two one-man vibrators, a flexible shaft model and a motor-in-head model.

The flexible shaft vibrator, with a 1½-hp electric motor, features a twin variable eccentric that allows for variation of amplitude and frequency according to job requirements. The motor-in-head model features removable motor section that allows for prompt replacement. It operates on 110 v ac or dc.—Champion Manufacturing Co., 3700 Forest Park Ave., St. Louis 8, Mo.

Epoxy-based coating

An epoxy-based combination skid-proofing and protective coating, called PG-1013, has been developed for concrete factory floors. The company says the compound will literally weld itself to concrete surfaces, sealing cracks, waterproofing, and providing corrosion protection, and will also bond tenaciously to masonry, steel, and many other building materials. A small percentage of mineral aggregate in the compound provides skid resistance. The coating is available in 1- and 10-gal. units in gray, red, or green.—Permagile Corp. of America, 34-43 56th St., Woodside 77, N. Y.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

PENNTROWEL SURFACING COMPOUNDS (Data Sheet CP-630)—Describes three Penntrowel fillers: Penntrowel Regular, Penntrowel Carbon, and Penntrowel Floor Surfacing, intended for corrosion and wear protection. Contains data on physical properties, application instruction, container and shipping information, and estimating.

APPLICATION INSTRUCTIONS FOR PENNTROWEL SURFACING COMPOUNDS (Data Sheet CP-631)—Information on how to figure quantities of materials, surface preparation, mixing procedure (illustrated by photographs), application, repairs, and safety precautions.—Pennsalt Chemicals Corp., Corrosion Engineering Products Dept., Natrona, Pa.

1958 HUMES CATALOG—Completely indexed, 185-page catalog illustrates and describes various products manufactured by Humes, Ltd., including a wide range of concrete pipes; concrete products, both precast and prestressed; steel pipes; pipe coatings; and linings.—Humes, Ltd., 185 William St., P. O. Box 4534, Melbourne, Australia

LOW-COST FLY ASH HANDLING BY PNEUMATIC CONVEYOR (Bulletin FF-49-1)—Sixteen-page bulletin describes pneumatic systems used in fly ash handling, including the Airslide fluidizing conveyor, Fuller-Kinyon pumping system, and rotary compressor. Discusses typical installations in leading power companies and in a concrete block manufacturing process.—Fuller Co., Catasauqua, Pa.

PRECAST CONCRETE FLOOR AND ROOF SYSTEM—1959 Flexicore catalog includes photographs, design data, and detail drawings on the use of Flexicore with steel and reinforced concrete frames, plumbing, heating and cooling systems, and electrical wiring.—Flexicore Co., Inc., 1932 E. Monument Ave., Dayton 1, Ohio

PRODUCT DATA GUIDE (X-5A)—Bulletin gives information covering the use of major Master Builders concrete and masonry products. Includes data on estimating needs for and description of Pozzolith, Embeco, Masterplate, Anvil-Top, and others.—The Master Builders Co., Cleveland 3, Ohio

SPUN CONCRETE POLES AND PILES—A 20-page catalog giving general information on the manufacture and use of Betoma spun concrete poles and piling.

CENTRIFUGALLY SPUN PIPES—Catalog giving detailed information on Betoma spun pipes. Plants all over the world.—Betoma North America, Niagara Falls, Canada

PAINTING SWIMMING POOLS—National Swimming Pool Institute and National Paint, Varnish, and Lacquer Association have prepared a 12-page booklet for the use of swimming pool builders and owners. Topics covered include surface preparation for both concrete and metal pools, types of paint, methods of application, colors, and a section of miscellaneous hints.—National Paint, Varnish, and Lacquer Association, 1500 Rhode Island Ave., NW, Washington 5, D. C.

VIBRATING SCREENS (No. 2777)—48-page catalog presents Link-Belt's selection of vibrating screens, 12 different types in 212 sizes. Tables of materials, selection data, easy-to-read drawings, and cut-away photographs showing construction features are given. Also includes information on accessory items including screen cloth recommendations, data on bouncing ball decks, snubbing and plugging controls, and electrically heated screen cloths.—Link-Belt Co., Dept. PR, Prudential Plaza, Chicago 1, Ill.

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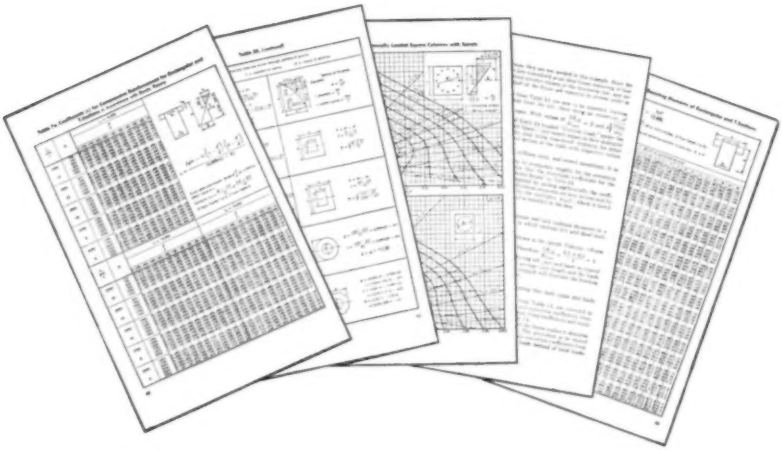

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